Seismic Evaluation of Institute Building NIT Rourkela

A THESIS SUBMITTED IN PARTIAL FULFILLMENT
OF THE REQUIREMENTS FOR THE DEGREE OF

Bachelor of Technology

In

Civil Engineering

By

Ankur Agrawal

Under the guidance of

Prof.A.V.Asha

Department of Civil Engineering
National Institute of Technology
Rourkela

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Certificate

This is to certify that the project entitled "**Seismic evaluation of institute building"** submitted by **Mr.Ankur Agrawal** [Roll No. 108CE032] in partial fulfillment of the requirements for the award of Bachelor of Technology degree in Civil engineering at the National Institute of Technology Rourkela (Deemed University) is an authentic work carried out by him under my supervision and guidance.

To the best of my knowledge the matter embodied in the project has not been submitted to any other university/institute for the award of any degree or diploma.

Date: 09th May 2012.

Prof. A.V.Asha
Department of Civil Engineering
National Institute of Technology
Rourkela- 769008

Acknowledgement

My heart pulsates with the thrill for tendering gratitude to those persons who helped in completion for my Project.

The pleasant point of presenting a report is the opportunity to thank those who have contributed to build my knowledge. Unfortunately, the list of expressions of thank no matter how extensive is always incomplete and inadequate. Indeed this page of acknowledgement shall never be able to touch the horizon of generosity of those who tendered their help to me.

I extend my deep sense of gratitude and indebtedness to my guide Prof. A. V. Asha for her kind attitude, keen interest, immense help, inspiration and encouragement which helped me carrying out the work. I an extremely grateful to Prof. R. Jha and Prof. S. K. Das for providing all kind of possible help throughout my project work.

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Ankur Agrawal

108CE032
B.Tech 8th Semester

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Abstract

There are many buildings which do not meet the current seismic requirement and suffer extensive damage during the earthquake. In 1960 when the institute building of NIT Rourkela was constructed, the seismic loading was not considered. The building is only deigned to take the dead and live loads. Evaluating the building for seismic conditions gives an idea whether the building is able to resist the earthquake load or not.

The objective is to evaluate an existing building for earthquake performance. Firstly preliminary evaluation is done and then detailed evaluation is carried out. For applying earthquake loads, equivalent static lateral force method is used according to IS 1893(Part 1):2002. The Demand Capacity Ratio (DCR) is carried out for beams and columns in order to evaluate the member for seismic loads. Since the reinforcement details of the building were not available as it is more than 50 years old, Design-1 is prepared applying only DEAD and LIVE loads according to IS 456:2000. This helps in estimating the reinforcement present in the building and in assuming that this much reinforcement is present. In Design-2 seismic loads are applied and from this demand obtained from design-2 and capacity from design -1, the DCR is calculated. If demand is more than capacity, the member fails and vice versa. STAAD-Pro V8i is used for loading and designing the building.

CHAPTER 1

INTRODUCTION

1.1General

The word earthquake is used to describe any seismic event whether natural or caused by humans that generates seismic waves. Earthquakes are caused mostly by rupture of geological faults, but also by other events such as volcanic activity, landslides, mine blasts, and nuclear tests. An earthquake (also known as a quake, tremor or temblor) is the result of a sudden release of energy in the Earth's crust that creates seismic waves. The seismicity or seismic activity of an area refers to the frequency, type and size of earthquakes experienced over a period of time. Earthquakes are measured using observations from seismometers. The moment magnitude is the most common scale on which earthquakes larger than approximately 5 are reported for the entire globe. The more numerous earthquakes smaller than magnitude 5 reported by national seismological observatories are measured mostly on the local magnitude scale, also referred to as the Richter scale.

There are many buildings that have primary structural system, which do not meet the current seismic requirements and suffer extensive damage during the earthquake. The buildings at NIT Rourkela were designed by primary structural system and the reason behind this is Rourkela lies in ZONE II of Seismic Zone Map of 2002 i.e. according to Seismic Zoning Map of IS:1893-2002, which says the region is least probable for earth quakes . The institute building is a four story building designed without considering the design factors of IS:1893-2002. At present time the methods for seismic evaluation of seismically deficient or earthquake damaged structures are not yet fully developed.

The buildings which do not fulfill the requirements of seismic design, may suffer extensive damage or collapse if shaken by a severe ground motion. The seismic evaluation reflects the seismic capacity of earthquake vulnerable buildings for the future use.

According to the Seismic Zoning Map of IS: 1893-2002,India is divided into four zones on the basis of seismic activities. They are Zone II, Zone III, Zone IV and Zone V. Rourkela lies in Zone II.

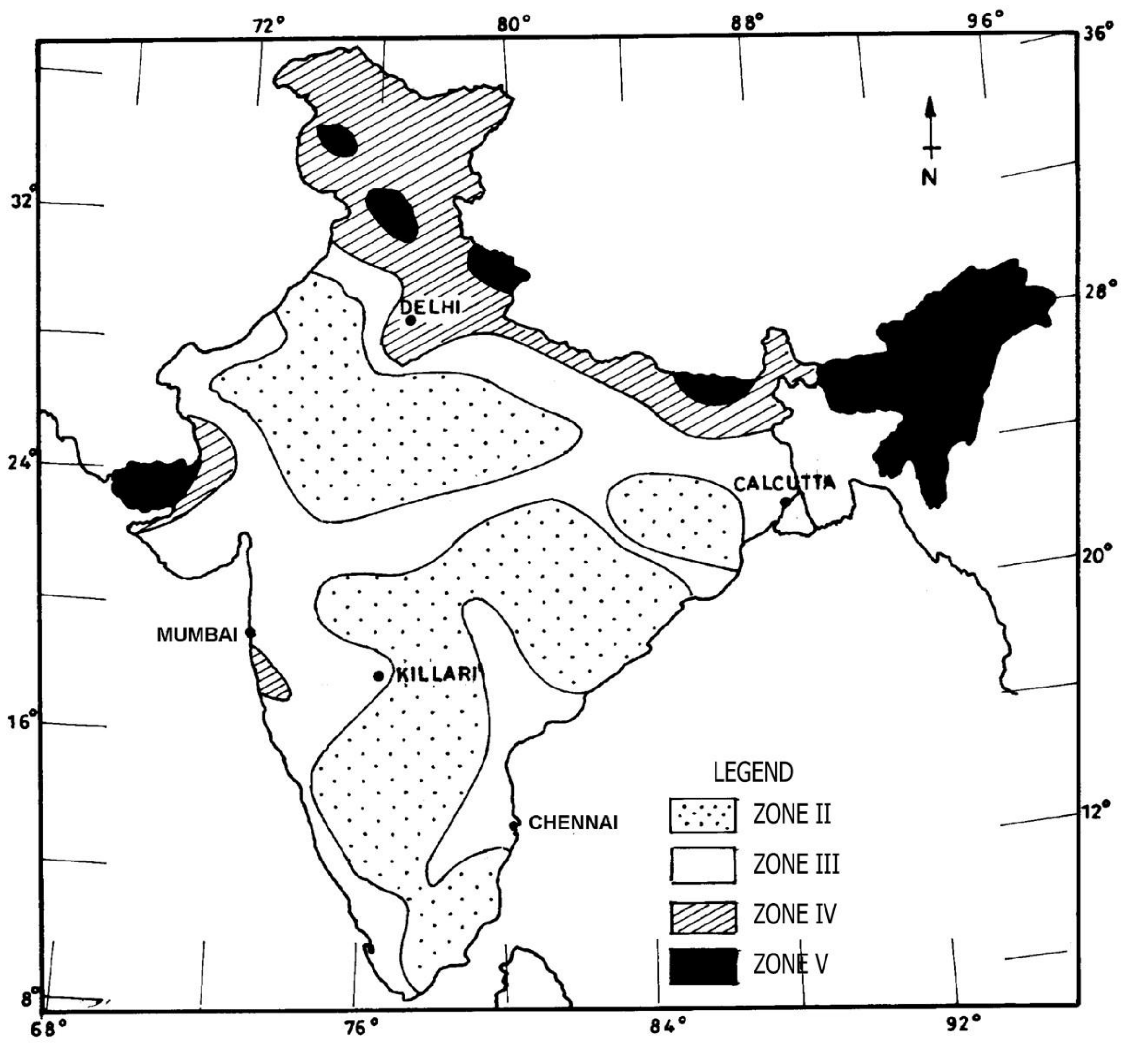


Fig 1.1: Seismic Zoning Map of India 2002

The methodologies available so far for the evaluation of existing buildings can be divided into two categories-(i) Qualitative method (ii) Analytical method.

The qualitative methods are based on the background information available of the building and its construction site, which require some or few documents like drawings, past performance of the similar buildings under seismic activities, visual inspection report and some non-destructive test results.The analytical methods are based on the consideration of the capacity and ductility of buildings on the basis of available drawings.

1.2 Proposed Work and Objective

My research project aims at evaluating the institute building of NIT Rourkela for seismic conditions. For designing Equivalent lateral force procedure is adopted.

The Demand Capacity Ratio (DCR) is carried out for beams and columns in order to evaluate the member for seismic loads. Since we did not find the reinforcement details of the building as it is more than 50 years old I have prepared Design-1 applying only DEAD and LIVE loads according to IS 456:2000 to estimate the reinforcement present in the building and assuming that this much reinforcement is present. In Design-2 seismic loads are applied and for this demand obtained from design-2 and capacity from design -1 the DCR is calculated. If demand is more than capacity member fails and vice versa.

The objective is to evaluate an existing building for earthquake performance.

Firstly preliminary evaluation is done and then detailed evaluation is carried out.

STAAD-Pro V8*i* is used for loading and designing the building.

1.3 Literature Review

Chandrasekaran and Rao (2002) investigated the design of multi- storied RCC buildings for seismicity. Reinforced concrete multi-storied buildings are very complex to model as structural systems for analysis. Usually, they are modeled as two-dimensional or three-dimensional frame systems using finite beam elements. However, no guidelines are available for the rational computation of sectional properties incorporating the effects of reinforcements in concrete members and the analysis is full of approximations. A case history of a RC structure, where the first author was involved in the study, is briefly cited in the paper. The current version of the IS: 1893 - 2002 requires that practically all multistoried buildings be analyzed as three-dimensional systems. This is due to the fact that the buildings have generally irregularities in plan or elevation or in both. Further, seismic intensities have been upgraded in weaker zones as compared to the last version IS: 1893-1984. It has now indirectly become mandatory to analyze all multistoried buildings in the country for seismic forces. This paper appraises briefly the significant changes in the current version of the code compared to the previous version. Some of the poor planning and construction practices of multistoried buildings in Peninsular India in particular, which lead to irregularities in plan and elevation of the buildings are also discussed in this paper. At present, there is too wide a variation in the modeling of buildings. This paper emphasises the need for guidelines in order to limit the range of assumptions to a narrow range. This is necessary to certify the analysis and design, or in case legal disputes arise later regarding the procedure adopted.

Shunsuke Otani (2004) studied earthquake resistant design of RCC Buildings (Past and Future). This paper briefly reviews the development of earthquake resistant design of buildings.

Measurement of ground acceleration started in 1930's, and the response calculation was made possible in 1940's. Design response spectra were formulated in the late 1950's to 1960's. Non-linear response was introduced in seismic design in 1960's and the capacity design concept was introduced in 1970's for collapse safety. The damage statistics of RCC buildings in 1995 Kobe disaster demonstrated the improvement of building performance with the development of design methodology. Buildings designed and constructed using outdated methodology should be upgraded. Performance basis engineering should be emphasized, especially for the protection of building functions following frequent earthquakes.

Durgesh C. Rai (2005) gave the guidelines for seismic evaluation and strengthening of buildings. This document is developed as part of project entitled "Review of Building Codes and Preparation of Commentary and Handbooks" awarded to Indian Institute of Technology Kanpur by the Gujarat State Disaster Management Authority (GSDMA), Gandhinagar through World Bank finances. This document is particularly concerned with the seismic evaluation and strengthening of existing buildings and it is intended to be used as a guide.

Abu Lego (2010) studied the Design of earthquake resistant building using Site Response spectra method. According to the Indian standard for Earthquake resistant design (IS: 1893), the seismic force depends on the zone factor (Z) and the average response acceleration coefficient (Sa/g) of the soil types at thirty meter depth with suitable modification depending upon the depth of foundation. In the present study an attempt has been made to generate response spectra using site specific soil parameters for some sites in seismic zone V, i.e. Arunachal Pradesh and Meghalaya and the generated response spectra is used to analyze some structures using commercial software STAAD Pro.

Saptadip Sarkar (2010) studies the Design of Earthquake resistant multi stories RCC building on a sloping ground which involves the analysis of simple 2-D frames of varying floor heights and varying no of bays using a very popular software tool STAAD Pro. Using the analysis results various graphs were drawn between the maximum axial force, maximum shear force, maximum bending moment, maximum tensile force and maximum compressive stress being developed for the frames on plane ground and sloping ground. The graphs used to drawn comparison between the two cases and the detailed study of “SHORT COLOUMN EFFECT” failure was carried up. In addition to that the detailed study of seismology was undertaken and the feasibility of the software tool to be used was also checked.

Chapter 2

Formulation

2.1 Formulation and Introduction:

The design philosophy adopted in the code is to ensure that structure possess minimum strength to resist minor earthquake, resist moderate earthquake and resist major earthquake. Actual forces on structures during earthquake are much higher than the design forces specified in the code.

The design lateral forces specified in the code shall be considered in each of the two orthogonal directions of the structure.

Procedure says the design base shear shall first be computed and then be distributed along the height of the buildings based on simple formulas appropriate for buildings with regular distribution of mass and stiffness. The design lateral force obtained at each floor level shall then be distributed to individual lateral load resisting elements depending upon floor diaphragm action.

The total shear in any horizontal plane shall be distributed to the various elements of lateral force resisting system on the basis of relative rigidity- Clause 7.7.2 of IS 1893(Part 1):2002.

The shear at any level depends on the mass at that level and deforms shape of the structure. Earthquake forces deflect a structure into number of shapes, known as the natural modes shapes.In equivalent lateral force procedure, the magnitude of lateral forces is based on the fundamental period of vibration. IS 1893 (Part1):2002 uses a parabolic distribution of lateral forces along the height of the building. In case of Institute building of NIT Rourkela, the deign lateral are applied by STAAD.Pro after feeding the required data in it.

The institute building of NIT Rourkela is a 4 storey RC framed structure. The building was constructed 50 years ago. The building mainly contains classrooms and academic offices. Due to the presence of construction joints and similar structures I have taken one four storey frame as my area of study. Since the reinforcement data was not available I have prepared Design 1 to estimate the reinforcement of the building using STAAD Pro and assume that this much reinforcement is present in the building. In design 1 only Dead Load and Live load is applied as it is assumed that it is not designed for earthquake load. For concrete design IS 456:2000 is followed. In design 2, in addition to dead load and live load seismic loads are also applied following IS 1893(part 1):2002. STAAD Pro V8*i* is used for designing purpose with full confidence on it. Supports are fixed.

2.2 Plan of building:

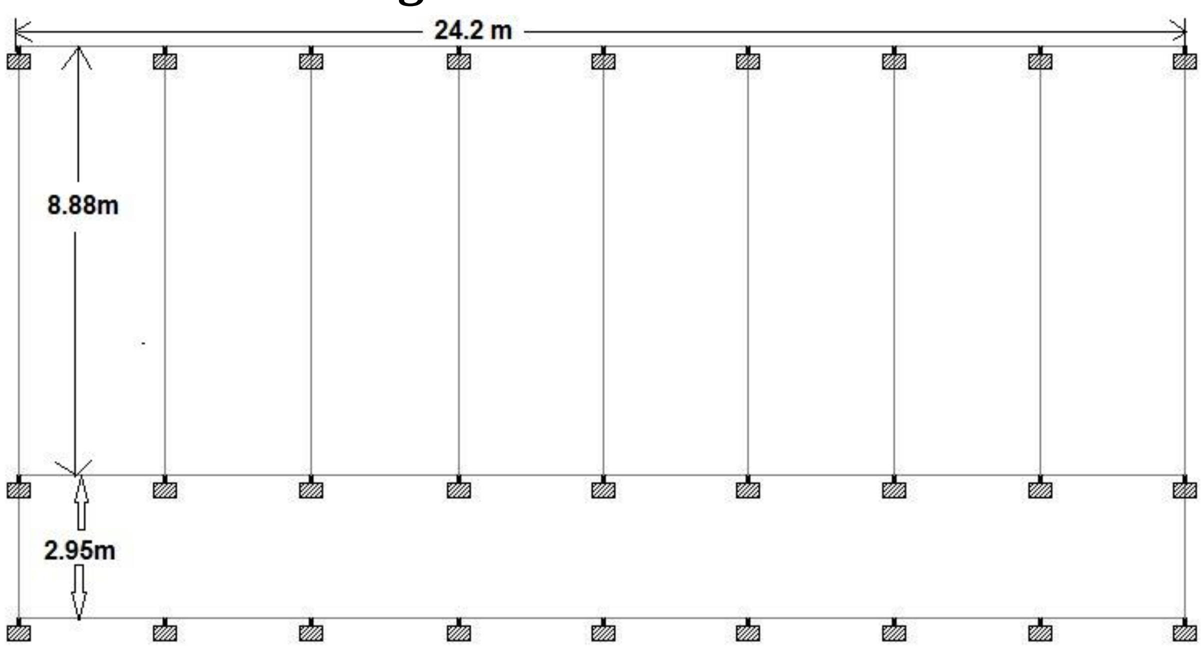


Fig 2.1: Plan of Building

Front Elevation:

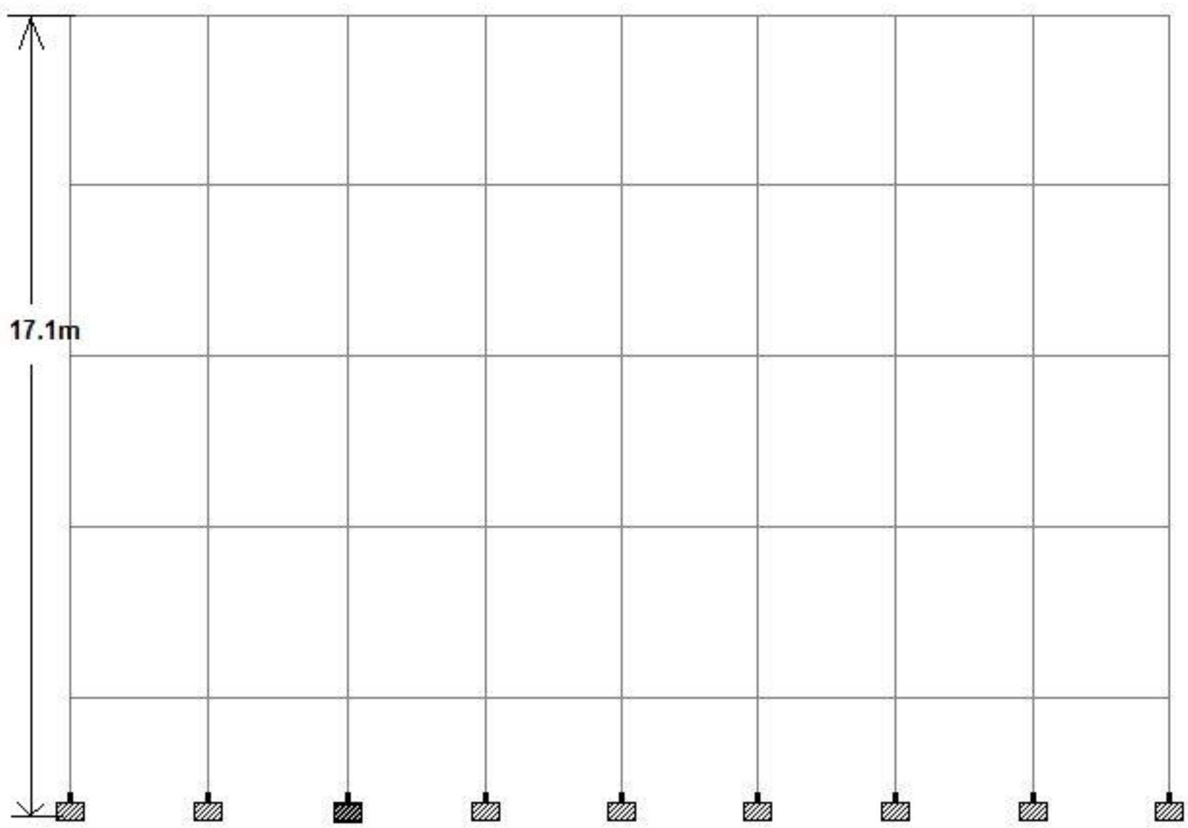
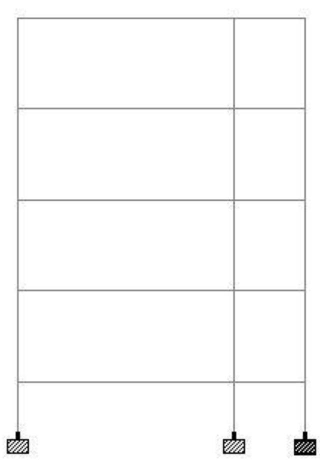


Fig 2.2: Front Elevation

Side View



***3-D* View:**

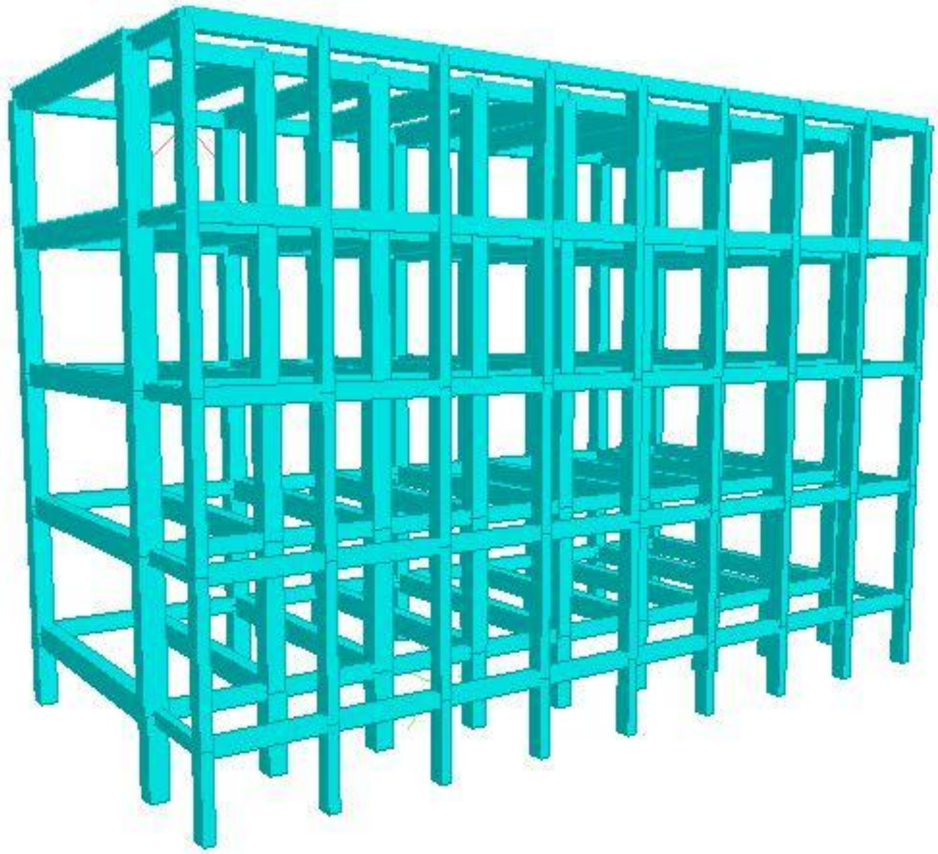


Fig 2.3: *3-D* View

2.3 Member:

<u>Beams:</u> A total of 140 beams with six different dimensions are present excluding the plinth beams.

Beam Type	Dimension (mm)
1	620 x 400
2	430 x 370
3	400 x 300
4	400 x 330
5	450 x 280
6	350 x 200

Table 2.1: Beam Dimensions

Columns: A total of 135 number of columns are present with four different dimensions.

Column Type	Dimension (mm)
1	470 x 570
2	280 x 450
3	520 x 550
4	340 x 550

Table 2.2: Column Dimensions

2.4 Design 1

2.4.1 Design Parameters:

- IS 456:2000 is followed.
- Grade of Concrete M15 implies F_{ck} =15 N/mm^2.
- Type of steel used – Mild Steel implies F_y =250 N/mm^2.
- Live Load = 1.5 KN/m^2 at roof (non-accessible)

  4 KN/m^2 at all other floors.
- Cover provided = 33mm for beams and 48mm for columns.
- Brick Load = 18.75 KN/m and 5KN/m.

2.4.2 Loading: Members are loaded with dead load and live load and as per

IS 875(Part 5) load combinations are applied.

Load Combinations-

- Dead Load
- Live Load
- 1.5 (Dead Load + Live Load)

Dead load on building:

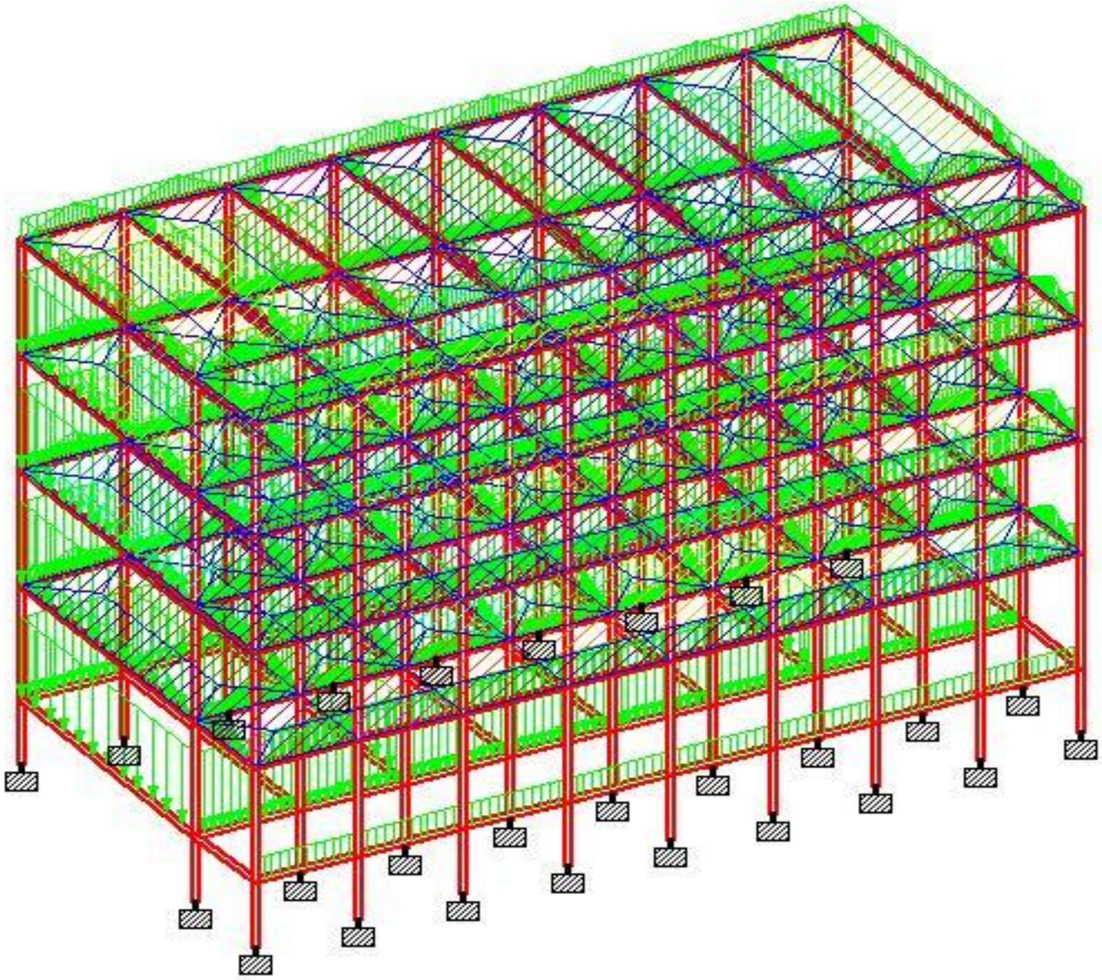


Fig.2.4: Dead Load on building

Dead load on first floor:

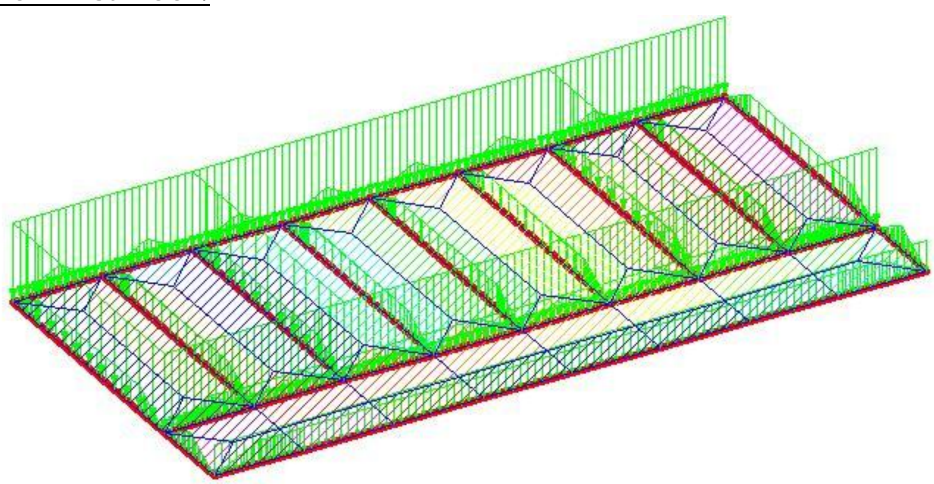


Fig 2.5 :Dead Load on First Floor

Live Load= 1.5 KN/m^2 on Roof and 4 KN/m^2 on all other floors.

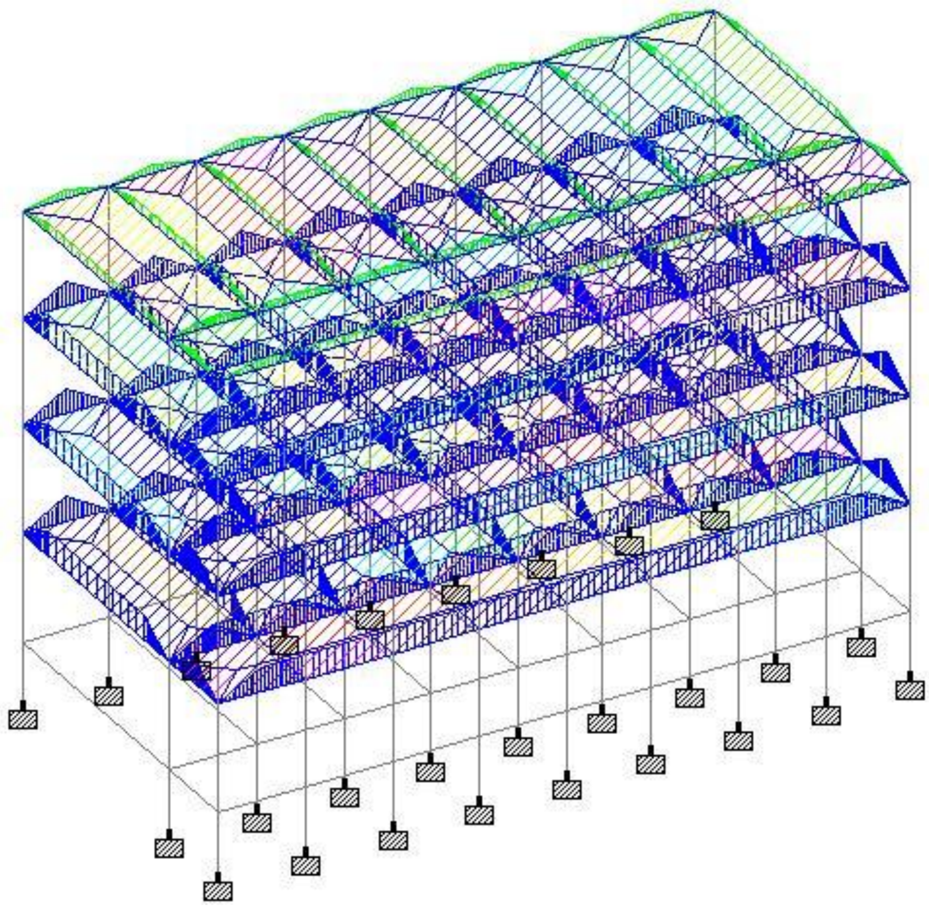


Fig: 2.6 Live Load On Building

Live Load on First Floor:

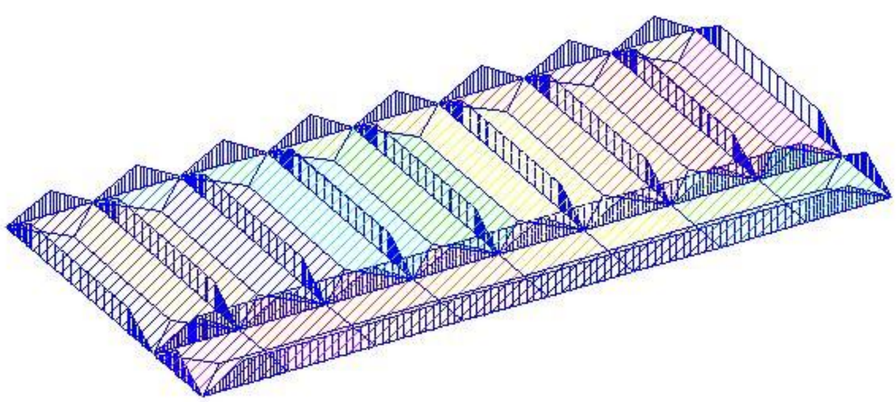


Fig 2.7 : Live Load on first floor

2.4.3 Results from Design 1:

Table 2.3 Area of Steel obtained from STAAD.Pro for beams of 1st floor

Beam	Top Reinforcement (sq.mm)			Bottom Reinforcement (sq.mm)		
No.	At Start	At Mid Span	At end	At Start	At Mid Span	At end
1	406.16	0	406.16	0	402.8	0
2	405.04	0	405.04	0	402.8	0
3	405.04	0	405.04	0	402.8	0
4	405.04	0	405.04	0	402.8	0
5	405.04	0	405.04	0	402.8	0
6	405.04	0	405.04	0	402.8	0
7	405.04	0	405.04	0	402.8	0
8	405.04	0	405.04	0	402.8	0
13	485.01	0	717.35	0	369.24	0
14	797.41	0	647.67	0	672.14	0
15	639.91	0	772.92	0	369.24	0
16	635.61	0	654.24	0	369.24	0
17	622.61	0	646.34	0	369.24	0
18	751.18	0	665.33	0	369.24	0
19	635.15	0	768.67	0	369.24	0
20	648.97	0	484.76	0	369.24	0
24	493.14	0	493.14	0	493.14	0
25	599.11	0	493.14	0	493.14	0
26	493.14	0	531.94	0	493.14	0
27	493.14	0	493.14	0	493.14	0
28	493.14	0	493.14	0	493.14	0
29	532.58	0	493.14	0	493.14	0
30	493.14	0	493.14	0	493.14	0
31	493.14	0	493.14	0	493.14	0
35	2499.84	0	2590.5	0	1529.22	0
36	4208.67	0	4238.95	1236.59	2866.31	1266.89
37	2409.22	0	2437.39	0	1564.78	0
38	2437.39	0	2448.31	0	1548.42	0
39	2452.98	0	2455.51	0	1558.39	0
40	4221.57	0	4211.45	1249.5	2859.5	1239.37
41	2458.36	0	2440.87	0	1551.93	0
42	1296.69	0	1332.62	0	926.46	0
23	3220.68	0	3312.87	189.45	2008.08	281.69
11	212.16	212.16	341.32	0	211.48	0
386	220.35	0	211.48	0	211.48	0

Table 2.4 Reinforcement obtained from STAAD.Pro for columns of Ground floor

Column	In mm^2 Area of Steel	Main Reinforcement	Tie Reinforcement
352	330.54	8 no.s ,12mm#@equal spacing	8 mm # @190mm c/c
353	524.27	8 no.s ,12mm#@equal spacing	8 mm # @190mm c/c
354	532.54	8 no.s ,12mm#@equal spacing	8 mm # @190mm c/c
355	525.78	8 no.s ,12mm#@equal spacing	8 mm # @190mm c/c
356	520.63	8 no.s ,12mm#@equal spacing	8 mm # @190mm c/c
357	523.95	8 no.s ,12mm#@equal spacing	8 mm # @190mm c/c
358	530.74	8 no.s ,12mm#@equal spacing	8 mm # @190mm c/c
359	517.68	8 no.s ,12mm#@equal spacing	8 mm # @190mm c/c
360	333.49	8 no.s ,12mm#@equal spacing	8 mm # @190mm c/c
363	2097.63	20 no.s , 12mm #@equal spacing	8 mm # @190mm c/c
364	1686.53	16 no.s , 12 mm #@equal spacing	8 mm # @190mm c/c
365	4300.13	40 no.s ,12 mm #@equal spacing	8 mm # @190mm c/c
366	1653.89	16 no.s , 12 mm #@equal spacing	8 mm # @190mm c/c
367	1628.04	16 no.s , 12 mm #@equal spacing	8 mm # @190mm c/c
368	1647.68	16 no.s , 12 mm #@equal spacing	8 mm # @190mm c/c
369	4304.54	40 no.s ,12 mm #@equal spacing	8 mm # @190mm c/c
370	1643.47	16 no.s , 12 mm #@equal spacing	8 mm # @190mm c/c
371	1022.38	12 no.s, 12 mm#@equal spacing	8 mm # @190mm c/c
374	1417.39	8 no.s , 16mm#@equal spacing	8 mm # @255mm c/c
375	1519.76	8 no.s, 16mm #@equal spacing	8 mm # @255mm c/c
376	3702.41	12 no.s, 20 mm#@equal spacing	8 mm # @255mm c/c
377	1480.41	8 no.s , 16mm#@equal spacing	8 mm # @255mm c/c
378	1451.45	8 no.s, 16mm #@equal spacing	8 mm # @255mm c/c
379	1476.84	8 no.s, 16mm #@equal spacing	8 mm # @255mm c/c
380	3720.74	12 no.s, 20 mm#@equal spacing	8 mm # @ 300mm c/c
381	1475.87	8 no.s , 16mm#@equal spacing	8 mm # @ 255mm c/c
382	896.75	8 no.s, 12mm#@equal spacing	8 mm # @ 190mm c/c

Results are obtained from STAAD.Pro for all the members. For beam no.24 the results are shown below:

Section of Beam No. 24 :

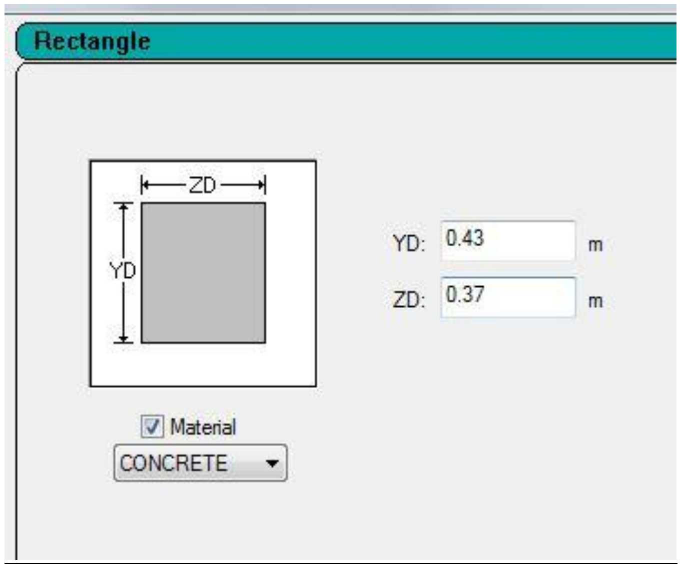


Fig 2.8: Section of a beam

Reinforcement details of Beam No. 24 from STAAD.Pro.

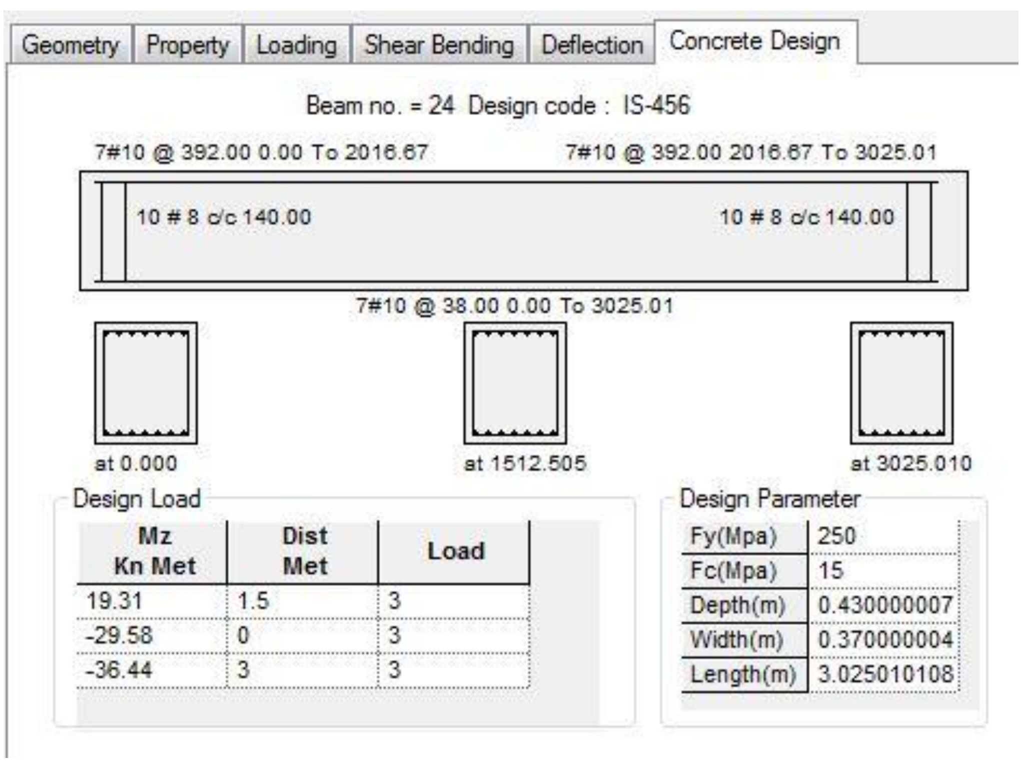


Fig. 2.9: Concrete Design of a beam Design 1

Shear Bending of Beam No. 24

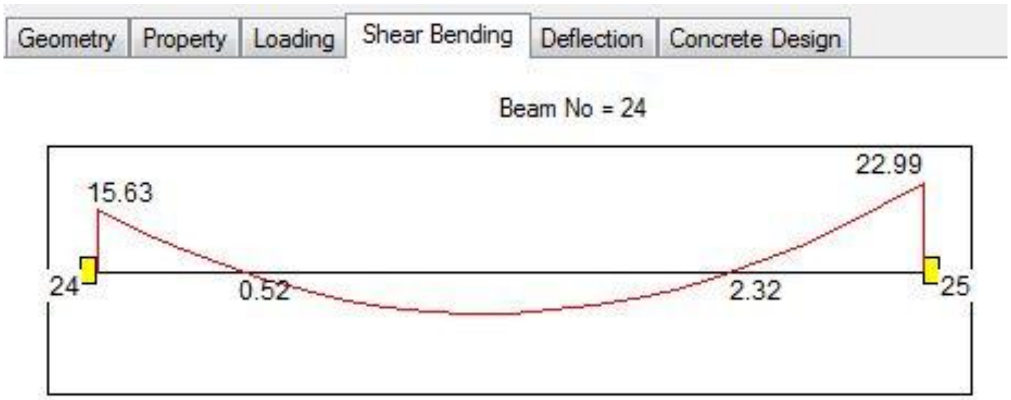


Fig. 2.10: Shear bending of a beam Design 1

Beam force Details of beam No. 24

Table 2.5 : Beam force details of a beam Design 1

All / Max Axial Forces / Max Bending Moments / Max Shear Forces

Beam	L/C	Dist m	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
24	1 DEAD LOAD	0.000	-0.247	36.176	0.064	1.743	-0.104	15.633
		0.756	-0.247	17.931	0.064	1.743	-0.056	-4.960
		1.513	-0.247	-2.431	0.064	1.743	-0.007	-10.954
		2.269	-0.247	-22.793	0.064	1.743	0.042	-1.283
		3.025	-0.247	-41.039	0.064	1.743	0.090	22.987
	2 LIVE LOAD	0.000	-0.634	5.494	-0.034	-0.923	0.050	4.085
		0.756	-0.634	4.350	-0.034	-0.923	0.024	0.219
		1.513	-0.634	0.919	-0.034	-0.923	-0.001	-1.918
		2.269	-0.634	-2.513	-0.034	-0.923	-0.027	-1.171
		3.025	-0.634	-3.657	-0.034	-0.923	-0.053	1.306
	3 COMBINATION LOAD CASE 3	0.000	-1.321	62.506	0.046	1.229	-0.081	29.577
		0.756	-1.321	33.422	0.046	1.229	-0.047	-7.112
		1.513	-1.321	-2.269	0.046	1.229	-0.013	-19.308
		2.269	-1.321	-37.959	0.046	1.229	0.022	-3.681
		3.025	-1.321	-67.043	0.046	1.229	0.056	36.439

Shear Bending of Column No.374 :

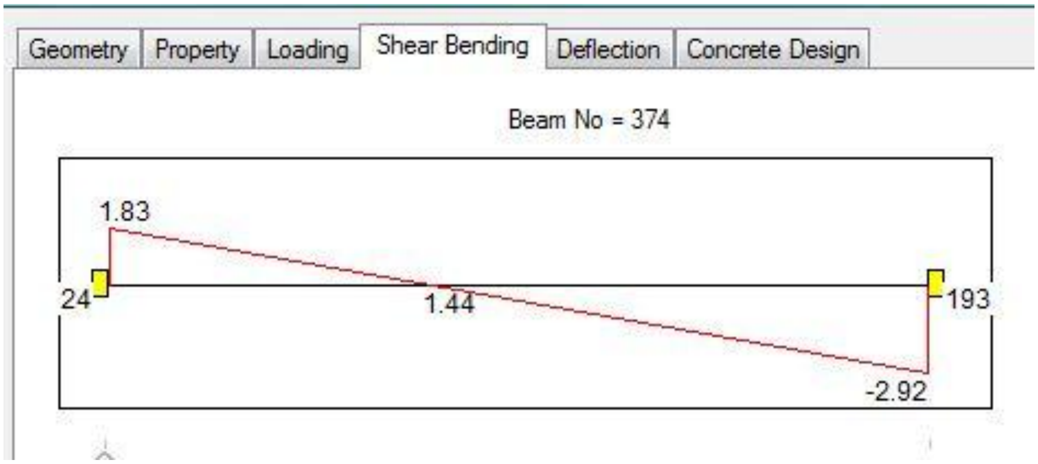


Fig.2.11:Shear Bending of column Design 1

Concrete Design of Column 374:

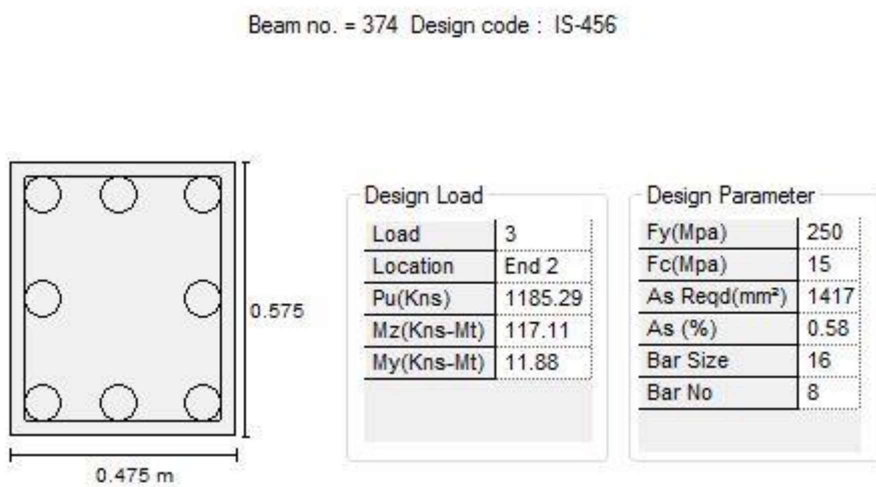


Fig.2.12 Concrete Design of Column Design 1

The above results are obtained for all the beams and columns. Only first floor beams are shown because these beams are taken under study for seismic evaluation. The result obtained from both the designs is used in chapter 3.

2.5 Design 2:

Design 2- to get the reinforcement after applying seismic loads resulting that if the building would have been designed for earthquake loads at least this much reinforcement should be present.

2.5.1 Design Parameters

- IS 456:2000 is followed.
- Grade of Concrete M15 implies F_{ck} =15 N/mm^2.
- Type of steel used – Mild Steel implies F_y =250 N/mm^2.
- Live Load = 1.5 KN/m^2 at roof (non-accessible)

  4 KN/m^2 at all other floors.
- Cover provided = 33mm for beams and 48mm for columns.
- Brick Load = 18.75 KN/m and 5KN/m.

2.5.2 Defining Seismic Load: (As per IS 1893 (Part 1):2002)

1. Zone Factor (Z) : Z= 0.1

   It is a factor to obtain a design spectrum depending on the perceived maximum risk characterized by maximum considered earthquake (MCE) in the zone in which structure is located. Zone factor is given in ***Table 2 of IS 1893 (Part 1):2002*** .Z can also be determined from the seismic zone map of India, shown in ***figure 1 of IS 1893 (Part 1):2002***.

2. Response Reduction Factor (R) : R= 3

   It is the factor by which actual base shear force, that would be generated if the structure were to remain elastic during its response to the design basis earthquake shaking ,shall be reduced to obtain the designed lateral force. The value of R is given in ***Table 7 of IS 1893 (Part 1):2002***.

3. Importance Factor (I) : I = 1.5

   It is a factor used to obtain the design seismic force depending upon the functional use of the structure. The minimum values of I are given in ***Table 6 of IS 1893 (Part 1):2002.***

4. Time Period (T): T = 0.63 seconds.

   The fundamental natural periods for buildings are given in ***Clause 7.6 of IS 1893(Part 1):2002.*** For RC framed buildings it is

$$\mathbf{T_a = 0.075h^{0.75}}$$

5. $S_{a/g}$ = 1.078
   Average Response acceleration coefficient for rock and soil sites as given by ***Figure 2 of IS 1893 (Part 1):2002.***

6. Damping = 5%.

7. Depth of Foundation = 1.5m.

8. Base Shear = 499.3KN (From STAAD.Pro)

   As per clause 7.5 of IS 1893(Part 1):2002. Base shear is calculated as:

$$V_B = A_h W$$

Where,

A_h = Design horizontal seismic coefficient for a structure.

$$A_h = (Z/2)(I/R)(S_a/g)$$

W= Seismic Weight of the building.

9. Soil Site Factor (SS) = 2 for medium soil taken from STAAD.Pro .

2.5.3 Loading:

In addition to dead load and live load , seismic loads and their load combinations are applied as per IS 1893 (Part 1):2002.

Load Combinations:

1. Seismic load in X direction (SX)
2. Seismic load in Z direction (SZ)
3. Dead Load (DL)
4. Live Load (LL)
5. Load combination (DL+LL)
6. Load Combination (1.5 SX +0.9 DL)
7. Load Combination (-1.5 SX + 0.9 DL)
8. Load Combination (1.5 SZ + 0.9 DL)
9. Load Combination (-1.5 SZ + 0.9 DL)
10. Load Combination (1.2SX+1.2DL+1.2LL)
11. Load Combination (1.2 SZ+1.2DL+1.2LL)
12. Load Combination (-1.2 SX+1.2DL+1.2LL)
13. Load Combination (-1.2 SZ+1.2DL+1.2LL)

14. Load Combination 1.5(DL+LL)

15. Load Combination (1.5SX+1.5 DL)

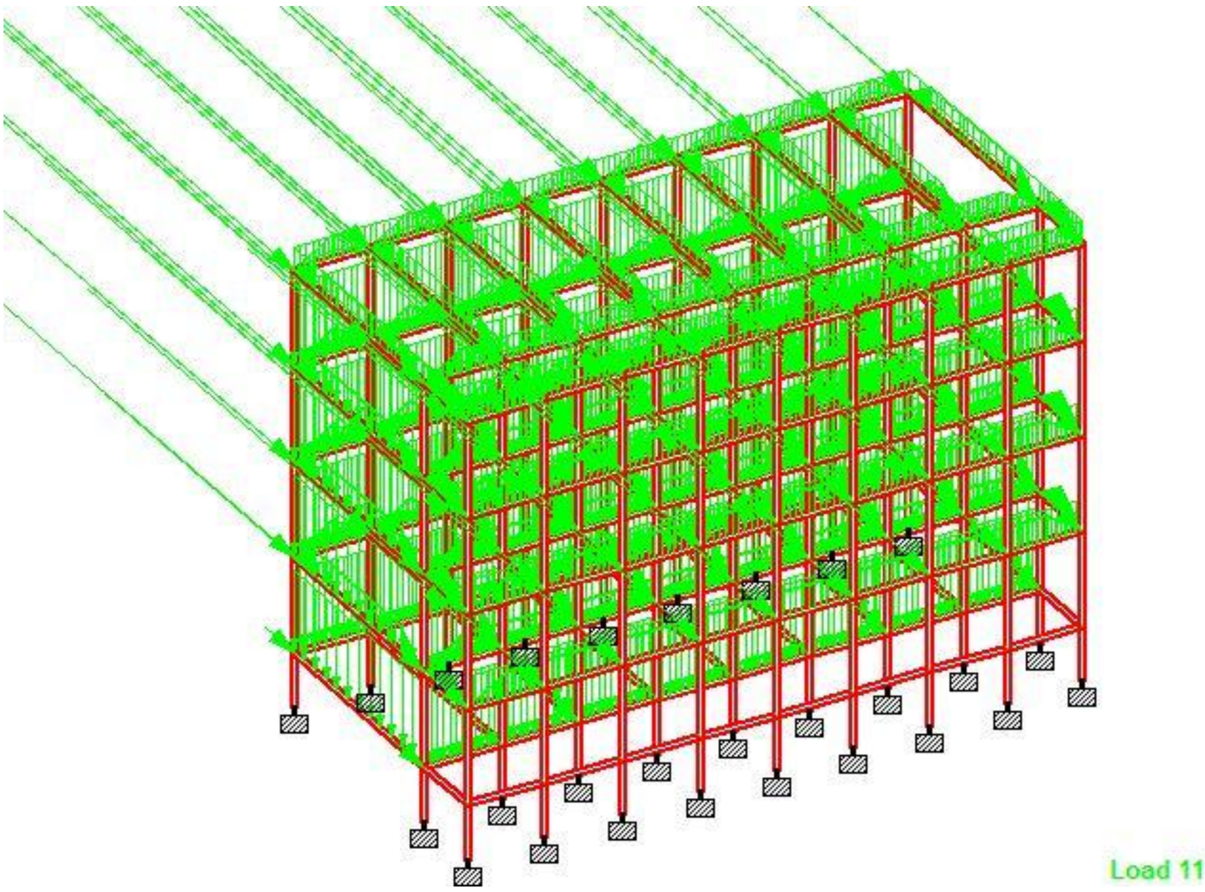
16. Load Combination (-1.5 SX+1.5 DL)

17. Load Combination (1.5 SZ+1.5 DL)

18. Load Combination (-1.5 SZ+1.5DL)

Got F_X, F_Y, F_Z, M_X, M_Y, M_Z and reinforcement for these load combinations.

Loading for load combination 11



Loading for combination No.11

Seismic Load on the building:

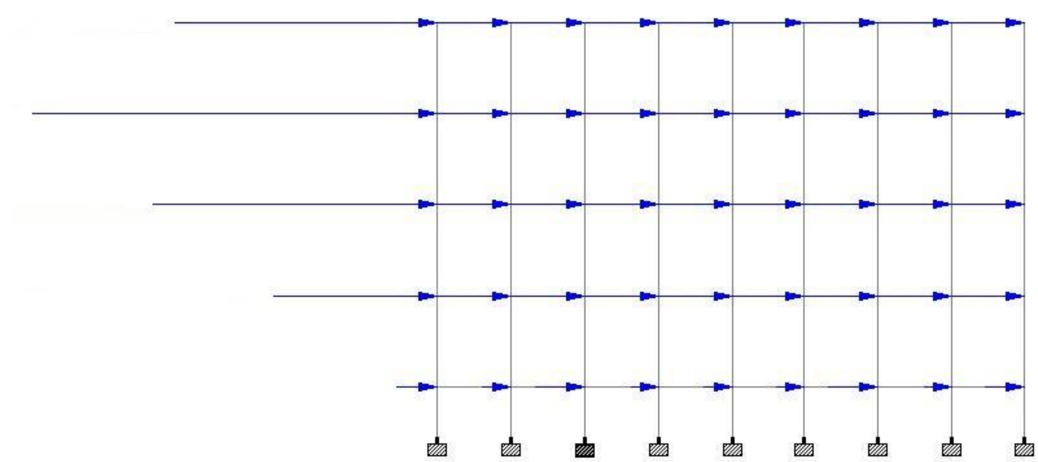


Fig. 2.13: Seismic Load on the building

2.5.4 Results from Design 2:

Table 2.6: Area of Steel obtained from STAAD.Pro for beams of 1st floor

Beam No.	Top Reinforcement (in Sq.mm)			Bottom Reinforcement (in Sq.mm)		
	At Start	At Mid Span	At End	At Start	At Mid Span	At End
1	621.6	405.04	612.07	402.8	402.8	402.8
2	566.06	402.8	593.21	402.8	402.8	402.8
3	585.53	0	594.48	402.8	402.8	402.8
4	581.61	402.8	588.2	402.8	402.8	402.8
5	577.11	402.8	589.5	402.8	402.8	402.8
6	582.93	402.8	593.98	402.8	402.8	402.8
7	580.48	402.8	571.38	402.8	402.8	402.8
8	605.61	405.04	631.51	402.8	402.8	402.8
13	1652.48	0	1724.76	773.97	368.22	583.64
14	1733.84	368.22	1613.95	578.66	373.2	668.47
15	1582.01	0	1682.45	651.03	368.22	512.33
16	1501.1	369.24	1521.18	530.57	369.24	502.52
17	1496.33	0	1520.3	353.58	369.24	506.98
18	1661.52	368.22	1609.84	554.42	368.22	638.74
19	1586.55	0	1693.41	660.53	368.22	522.05
20	1559.04	0	1252.41	504.87	398.2	585.87
24	1620.56	491.88	1730.17	941.27	486.85	12.54
25	1762.3	491.88	1501.01	690.43	491.88	873.6
26	1540.12	493.14	1679.75	843.97	493.14	650.6

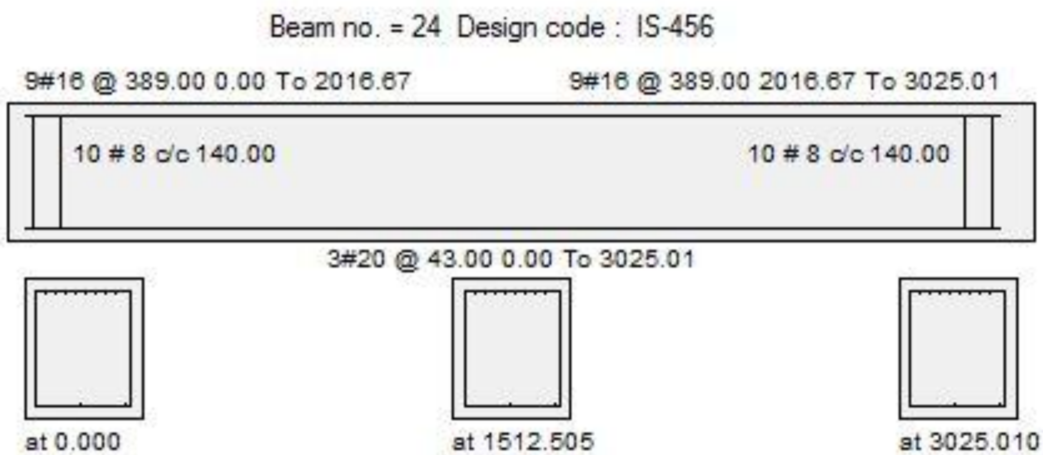
27	1361.23	491.88	1373.91	627.09	491.88	627.75
28	1361.26	491.88	1367.12	640.85	491.88	629.49
29	1594.16	493.14	1492.67	653.43	493.14	840.7
30	1547.13	0	1706.78	852.3	493.14	658.88
31	1495.08	0	1160.45	559.95	493.14	681.91
35	3119.33	0	3168.46	784.72	1529.19	784.72
36	4396.6	781.32	4424.72	1424.62	2866.22	1452.75
37	3046.11	0	3062.99	784.72	1564.76	784.72
38	3031.26	787.44	3038.3	784.72	1548.4	784.72
39	3046.02	787.44	3048.44	784.72	1558.36	784.72
40	4405.76	781.32	4399.8	1433.79	2859.06	1427.82
41	3069.49	787.44	3056.81	784.72	1545.31	781.72
42	2072.59	788.43	2009.08	791.52	828.37	791.52
23	3807.04	0	3953.6	794.58	2008.02	947.22
11	743.31	121.16	858.77	553.81	212.06	378.81
386	534.77	212.16	824.3	518.01	212.06	462.88

Table 2.7:Reinforcement obtained from STAAD.Pro for columns of Ground Floor

Column	Area of Steel in Sq.mm	Main Reinforcement	Tie Reinforcement
352	990	12 no.s , 12mm #	8mm # @190mm c/c
353	697.9	8 no.s ,12mm#	8mm # @190mm c/c
354	990	12 no.s , 12mm #	8mm # @190mm c/c
355	990	12 no.s , 12mm #	8mm # @190mm c/c
356	990	12 no.s , 12mm #	8mm # @190mm c/c
357	990	12 no.s , 12mm #	8mm # @190mm c/c
358	990	12 no.s , 12mm #	8mm # @190mm c/c
359	990	12 no.s , 12mm #	8mm # @190mm c/c
360	422.58	8 no.s ,12mm#	8mm # @190mm c/c
363	3600.65	32 no.s , 12mm #	8mm # @190mm c/c
364	1686.49	16 no.s , 12mm #	8mm # @190mm c/c
365	4300.15	40 no.s , 12mm #	8mm # @190mm c/c
366	1653.94	16 no.s , 12mm #	8mm # @190mm c/c
367	1980	20 no.s , 12 mm#	8mm # @190mm c/c
368	1980	20 no.s , 12 mm#	8mm # @190mm c/c
369	4303.56	40 no.s , 12mm #	8mm # @190mm c/c
370	1658	16 no.s , 12mm #	8mm # @190mm c/c
371	2849.15	16 no.s , 16mm #	8mm # @255mm c/c
374	2808.65	28 no.s , 12mm #	8mm # @190mm c/c

375	2185	20 no.s , 12 mm#	8mm # @190mm c/c
376	3702.58	12 no.s , 12mm #	8mm # @300mm c/c
377	1980	12 no.s , 20mm #	8mm # @190mm c/c
378	1980	20 no.s , 12 mm#	8mm # @190mm c/c
379	1980	20 no.s , 12 mm#	8mm # @190mm c/c
380	3719.16	12 no.s ,20mm #	8mm # @300mm c/c
381	1980	20 no.s , 12 mm#	8mm # @190mm c/c
382	2661.78	24 no.s , 12mm #	8mm # @190mm c/c

Reinforcement Details of Beam 24 from STAAD.Pro



Design Load

Mz Kn Met	Dist Met	Load
68.2	0	6
-104.77	0	16
-109.26	3	15

Design Parameter

Fy(Mpa)	250
Fc(Mpa)	15
Depth(m)	0.430000007
Width(m)	0.370000004
Length(m)	3.025010108

Fig.2.14: Concrete Design of beam Design 2

Concrete Design of Column 374 :

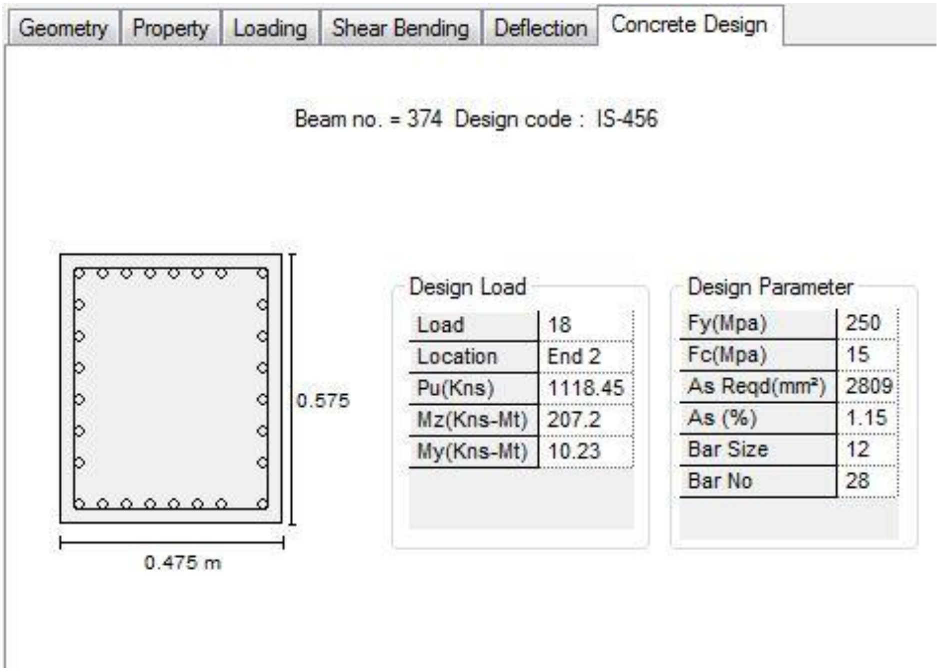


Fig.2.15 : Concrete Design of column Design 2

Beam force details of Beam 24:

Table 2.8 : Beam force details of beam

Beam	L/C	Dist m	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
24	1 SL IN X	0.000	-1.733	-34.414	-0.253	0.144	0.507	-54.610
		0.756	-1.733	-34.414	-0.253	0.144	0.315	-28.584
		1.513	-1.733	-34.414	-0.253	0.144	0.123	-2.558
		2.269	-1.733	-34.414	-0.253	0.144	-0.068	23.468
		3.025	-1.733	-34.414	-0.253	0.144	-0.260	49.494
	2 SL IN Z	0.000	-0.254	0.089	0.311	0.294	-0.530	0.148
		0.756	-0.254	0.089	0.311	0.294	-0.295	0.081
		1.513	-0.254	0.089	0.311	0.294	-0.060	0.013
		2.269	-0.254	0.089	0.311	0.294	0.175	-0.054
		3.025	-0.254	0.089	0.311	0.294	0.411	-0.122
	3 DEAD LOA	0.000	-0.207	35.927	0.064	1.740	-0.104	15.238
		0.756	-0.207	17.681	0.064	1.740	-0.055	-5.166
		1.513	-0.207	-2.680	0.064	1.740	-0.007	-10.972
		2.269	-0.207	-23.042	0.064	1.740	0.041	-1.112
		3.025	-0.207	-41.288	0.064	1.740	0.090	23.346
	4 LIVE LOAD	0.000	-0.634	5.494	-0.034	-0.923	0.050	4.085
		0.756	-0.634	4.350	-0.034	-0.923	0.024	0.219
		1.513	-0.634	0.919	-0.034	-0.923	-0.001	-1.918
		2.269	-0.634	-2.513	-0.034	-0.923	-0.027	-1.171
		3.025	-0.634	-3.657	-0.034	-0.923	-0.053	1.306

Beam	L/C	Dist m	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
	5	0.000	-0.841	41.421	0.030	0.817	-0.054	19.323
		0.756	-0.841	22.032	0.030	0.817	-0.031	-4.948
		1.513	-0.841	-1.762	0.030	0.817	-0.008	-12.890
		2.269	-0.841	-25.555	0.030	0.817	0.014	-2.283
		3.025	-0.841	-44.945	0.030	0.817	0.037	24.652
	6	0.000	-2.786	-19.287	-0.323	1.782	0.667	-68.200
		0.756	-2.786	-35.708	-0.323	1.782	0.423	-47.525
		1.513	-2.786	-54.034	-0.323	1.782	0.179	-13.711
		2.269	-2.786	-72.360	-0.323	1.782	-0.065	34.202
		3.025	-2.786	-88.781	-0.323	1.782	-0.309	95.253
	7	0.000	2.414	83.956	0.438	1.351	-0.854	95.628
		0.756	2.414	67.535	0.438	1.351	-0.523	38.226
		1.513	2.414	49.209	0.438	1.351	-0.191	-6.038
		2.269	2.414	30.884	0.438	1.351	0.140	-36.204
		3.025	2.414	14.462	0.438	1.351	0.471	-53.230
	8	0.000	-0.567	32.468	0.524	2.007	-0.888	13.936
		0.756	-0.567	16.047	0.524	2.007	-0.492	-4.529
		1.513	-0.567	-2.279	0.524	2.007	-0.096	-9.855
		2.269	-0.567	-20.604	0.524	2.007	0.301	-1.082
		3.025	-0.567	-37.026	0.524	2.007	0.697	20.829
	9	0.000	0.195	32.201	-0.409	1.125	0.702	13.492
		0.756	0.195	15.780	-0.409	1.125	0.392	-4.771
		1.513	0.195	-2.546	-0.409	1.125	0.083	-9.895
		2.269	0.195	-20.872	-0.409	1.125	-0.226	-0.920
		3.025	0.195	-37.293	-0.409	1.125	-0.535	21.194
	10	0.000	-3.089	8.408	-0.268	1.153	0.544	-42.344
		0.756	-3.089	-14.859	-0.268	1.153	0.341	-40.238
		1.513	-3.089	-43.411	-0.268	1.153	0.138	-18.537
		2.269	-3.089	-71.963	-0.268	1.153	-0.065	25.422
		3.025	-3.089	-95.231	-0.268	1.153	-0.267	88.976

Beam	L/C	Dist m	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
	11	0.000	-1.314	49.813	0.409	1.333	-0.701	23.365
		0.756	-1.314	26.545	0.409	1.333	-0.391	-5.841
		1.513	-1.314	-2.007	0.409	1.333	-0.082	-15.452
		2.269	-1.314	-30.559	0.409	1.333	0.228	-2.805
		3.025	-1.314	-53.827	0.409	1.333	0.537	29.436
	12	0.000	1.071	91.003	0.340	0.808	-0.673	88.719
		0.756	1.071	67.735	0.340	0.808	-0.416	28.363
		1.513	1.071	39.183	0.340	0.808	-0.158	-12.399
		2.269	1.071	10.631	0.340	0.808	0.099	-30.902
		3.025	1.071	-12.636	0.340	0.808	0.356	-29.811
	13	0.000	-0.704	49.599	-0.337	0.627	0.571	23.010
		0.756	-0.704	26.331	-0.337	0.627	0.316	-6.034
		1.513	-0.704	-2.221	-0.337	0.627	0.062	-15.484
		2.269	-0.704	-30.773	-0.337	0.627	-0.193	-2.675
		3.025	-0.704	-54.041	-0.337	0.627	-0.448	29.728

Beam	L/C	Dist m	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
	14	0.000	-1.261	62.132	0.045	1.226	-0.081	28.984
		0.756	-1.261	33.048	0.045	1.226	-0.047	-7.422
		1.513	-1.261	-2.642	0.045	1.226	-0.013	-19.335
		2.269	-1.261	-38.333	0.045	1.226	0.022	-3.425
		3.025	-1.261	-67.417	0.045	1.226	0.056	36.978
	15	0.000	-2.910	2.269	-0.284	2.826	0.605	-59.058
		0.756	-2.910	-25.100	-0.284	2.826	0.390	-50.625
		1.513	-2.910	-55.642	-0.284	2.826	0.175	-20.294
		2.269	-2.910	-86.185	-0.284	2.826	-0.040	33.534
		3.025	-2.910	-113.554	-0.284	2.826	-0.255	109.261
	16	0.000	2.290	105.512	0.476	2.395	-0.916	104.771
		0.756	2.290	78.144	0.476	2.395	-0.556	35.126
		1.513	2.290	47.601	0.476	2.395	-0.196	-12.622
		2.269	2.290	17.058	0.476	2.395	0.164	-36.871
		3.025	2.290	-10.311	0.476	2.395	0.525	-39.222
	17	0.000	-0.691	54.024	0.562	3.052	-0.950	23.079
		0.756	-0.691	26.656	0.562	3.052	-0.525	-7.629
		1.513	-0.691	-3.887	0.562	3.052	-0.100	-16.438
		2.269	-0.691	-34.430	0.562	3.052	0.325	-1.750
		3.025	-0.691	-61.798	0.562	3.052	0.751	34.837
	18	0.000	0.071	53.757	-0.371	2.169	0.639	22.634
		0.756	0.071	26.388	-0.371	2.169	0.359	-7.871
		1.513	0.071	-4.154	-0.371	2.169	0.079	-16.478
		2.269	0.071	-34.697	-0.371	2.169	-0.201	-1.587
		3.025	0.071	-62.066	-0.371	2.169	-0.481	35.202

CHAPTER 3

Analysis

3.1 Evaluation

The purpose is to reduce loss of life and injury to in-habituating buildings which are constructed without earthquake resistant features. Analysis defines the minimum evaluation criteria for the expected performance of life- safety of existing buildings with appropriate modification to IS: 1893(Part 1):2002, which is applicable for the seismic design of new buildings. Reference is always made to the current edition of IS: 1893.All existing structural elements must be able to carry full other non-seismic loads in accordance with the current applicable codes related to loading and material strengths.

3.2 Definitions:

3.2.1Capacity: The permissible strength or deformation of a structural member or system.

3.2.2 Deformation: Relative displacement or rotation of the ends of a component or element or node.

3.2.3 Demand: The amount of force or deformation imposed on an element or component.

3.2.4 Lateral Force Resisting System:

The collection of frames, shear walls, bearing walls, braced frames and inter connecting horizontal diaphragms that provide earthquake resistance to a building.

3.2.5 Load Path:

The path that seismic forces acting anywhere in the building, take to the foundation of the structure and, finally, to the soil. Typically, load travel from the diaphragms through connections to the vertical lateral-force resisting elements, and then proceeds to the foundation.

3.2.6 Seismic Evaluation:

An approved process or methodology of evaluating deficiencies in a building which prevent the building from achieving life safety objective.

3.2.7 Weak Storey: The strength of the vertical lateral force resisting system in any storey shall not be less than 70% of the strength in an adjacent storey.

3.2.8 Soft Storey: The stiffness of vertical lateral load resisting system in any storey shall not be less than 60% of the stiffness in an adjacent storey or less than 70% of the average stiffness of the three storeys above.

(The above definitions are as per written in my reference no.- 9)

3.3 Preliminary Evaluation

Preliminary evaluation is a quick check the building for potential deficiencies and to assess the characteristics that can affect its vulnerability. It includes site visit, acceptability criteria, configuration related checks etc.

Criteria:

1. **Load Path:** The structure shall contain at least one rational and complete load

   path for seismic forces from any horizontal direction so that they can transfer all inertial forces in the building to the foundation.

   The provision is- The number of lines of vertical lateral load resisting elements in each principle direction shall be greater than or equal to 2.

As per my analysis the building satisfies the provision for load paths.

2. **Geometry:** Horizontal dimensions are equal at all stories.
3. **Weak and soft Storey:** No abrupt changes in column size from one storey to another hence, weak and soft storey does not exist.
4. **Vertical Discontinuities:** Vertical lateral force resisting elements are continuous to the foundation.
5. **Mass:** Effective mass at all the floors is equal except the roof.
6. **Short Column:** Short Column does not exist.

3.4 Detailed Evaluation

In detailed evaluation full building analysis is performed. The detailed evaluation procedure is based on the analysis and design philosophy of IS 1893 (Part1):2002.

This involves equivalent static lateral force procedure, load with response reduction factors and Demand Capacity Ratio (DCR) for ductility as in IS 13920.

Criteria:

The following checks are done :

1. Demand Capacity Ratio (DCR) for moments of resistance in sagging and hogging in case of beams.
2. DCR for Shear capacity in beams.
3. DCR for flexural capacity of column.
4. DCR for shear capacity of column.

3.4.1 Check for Beam

STEPS:

- Obtained the maximum moment induced on beam from Design 2.
- Calculated the capacity of members from the reinforcement obtained from Design 1.
- Demand capacity Ratio= Max. Moment/ Capacity.
- If the value of DCR<1 then the members is PASS i.e. it can take the moment induced by seismic loading.
- If the value of DCR>1 then the member is Fail i.e. it can't take the load due to earthquake.

Since the value of seismic load on First floor is least among all other floors, I have taken beams of first floor under study. If the beams of first floor fail we need not to check other floors because the seismic load is higher on other floors.

1. DCR for moments of resistance in Hogging and Sagging:

Calculation of moment of resistance in Hogging:

Moment of resistance (M.R)is calculated by using :

$$Mu=0.36f_{CK}\, b\, x\, (d-0.416\, x) + (f_{SC} - 0.44f_{CK})A_{SC}\, (d-d`)$$

Sample Calculation for BEAM 13:

F_{CK} = Characteristic strength of concrete = 15N/mm^2

F_Y = 250 N/mm^2 (Yield Strength of Steel)

Cover d'= 33mm

b = 300mm (Width)

d = 400mm (Depth)

A_{st} = 392.5 mm^2 (Area of steel in tension)

A_{sc} = 628 mm^2 (Area of steel in Compression)

x = 35.80 mm (Depth of Neutral Axis)

f_{sc} = 49.772 N/mm^2 (Stress in compression steel)

f_{st} = 217.5 N/mm^2

Xu/d = 0.098

Mu/bd^2 = 0.732

Implies M.R of the section = 58.086 KNm. (CAPACITY)

Demand from Design 2 = 29. 569 KNm

Demand Capacity Ratio(DCR) = Demand / Capacity = 101.59/ 29.569 = 3.4 (>1) => FAIL

Calculation of moment of resistance in Sagging (M.R):

For calculating moment of resistance in hogging

A_{st} =628 mm^2

And A_{sc} =392.5 mm^2

Giving M.R = 46.675 KNm (CAPACITY)

So, DCR = Demand/ Capacity

= 101.59/46.675

= 2.18 (DCR >1) => FAIL

Beams of First Floor:

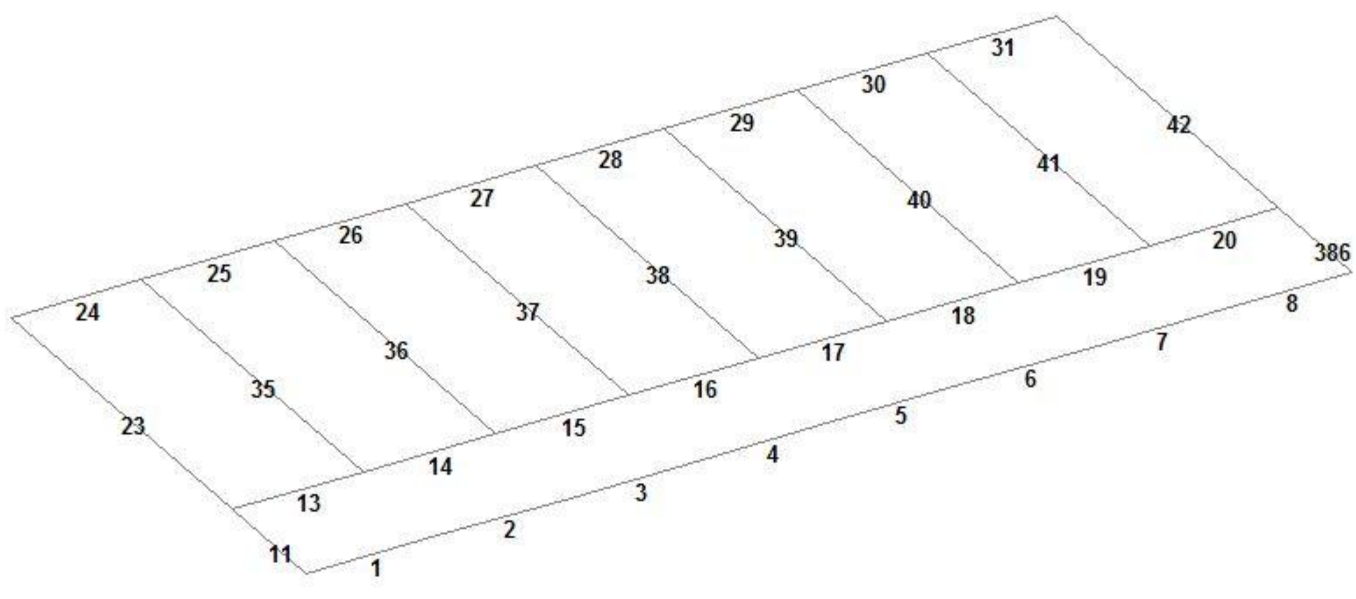


Fig.3.1: Beams of first floor

Numbers on members indicate the beam number.

Following the above procedure DCR value is calculated for all the beams of first floor and the results are tabulated below.

Table 3.1: Analysis result of beams of first floor

Beam NO.	Demand (KNm)	Capacity Sagging (KNm)	Capacity Hogging (KNm)	DCR Sagging	DCR Hogging	Result Sagging	Result Hogging
1	44.484	33.911	33.911	1.31	1.3	FAIL	FAIL
2	42.166	33.911	33.911	1.24	1.2	FAIL	FAIL
3	42.105	33.911	33.911	1.24	1.2	FAIL	FAIL
4	41.664	33.911	33.911	1.23	1.2	FAIL	FAIL
5	41.785	33.911	33.911	1.23	1.2	FAIL	FAIL
6	42.158	33.911	33.911	1.24	1.2	FAIL	FAIL
7	41.522	33.911	33.911	1.22	1.2	FAIL	FAIL
8	44.431	33.911	33.911	1.31	1.3	FAIL	FAIL
13	101.59	46.675	29.569	2.18	3.4	FAIL	FAIL
14	102.405	59.268	29.566	1.73	3.5	FAIL	FAIL
15	99.518	52.357	29.567	1.9	3.4	FAIL	FAIL
16	92.931	50.313	29.568	1.85	3.1	FAIL	FAIL
17	92.767	50.313	29.568	1.84	3.1	FAIL	FAIL
18	98.034	52.357	29.567	1.87	3.3	FAIL	FAIL
19	100.109	29.571	29.567	3.39	3.4	FAIL	FAIL
20	92.615	29.571	29.571	3.13	3.1	FAIL	FAIL

24	109.261	44.816	44.816	2.44	2.4	FAIL	FAIL
25	112.292	44.815	44.815	2.51	2.5	FAIL	FAIL
26	106.209	44.816	44.816	2.37	2.4	FAIL	FAIL
27	97.311	44.816	44.816	2.17	2.2	FAIL	FAIL
28	97.158	44.816	44.816	2.17	2.2	FAIL	FAIL
29	105.714	44.816	44.816	2.36	2.4	FAIL	FAIL
30	107.219	44.816	44.816	2.39	2.4	FAIL	FAIL
31	97.257	44.816	44.816	2.17	2.2	FAIL	FAIL
35	306.418	311.84	190.597	0.98	1.6	PASS	FAIL
36	448.541	521.15	521.152	0.86	0.9	PASS	PASS
37	294.079	294.41	190.599	1	1.5	PASS	FAIL
38	291.341	294.41	190.599	0.99	1.5	PASS	FAIL
39	292.528	300.91	190.598	0.97	1.5	PASS	FAIL
40	446.49	521.16	521.155	0.86	0.9	PASS	PASS
41	294.893	300.91	190.598	0.98	1.5	PASS	FAIL
42	105.72	114.93	114.927	0.92	1.1	PASS	FAIL
23	400.526	404.47	243.568	0.99	1.6	PASS	FAIL
11	44.328	14.669	14.669	3.02	3	FAIL	FAIL
386	42.932	14.669	14.669	2.93	2.9	FAIL	FAIL

Beams Pass Sagging Moment:

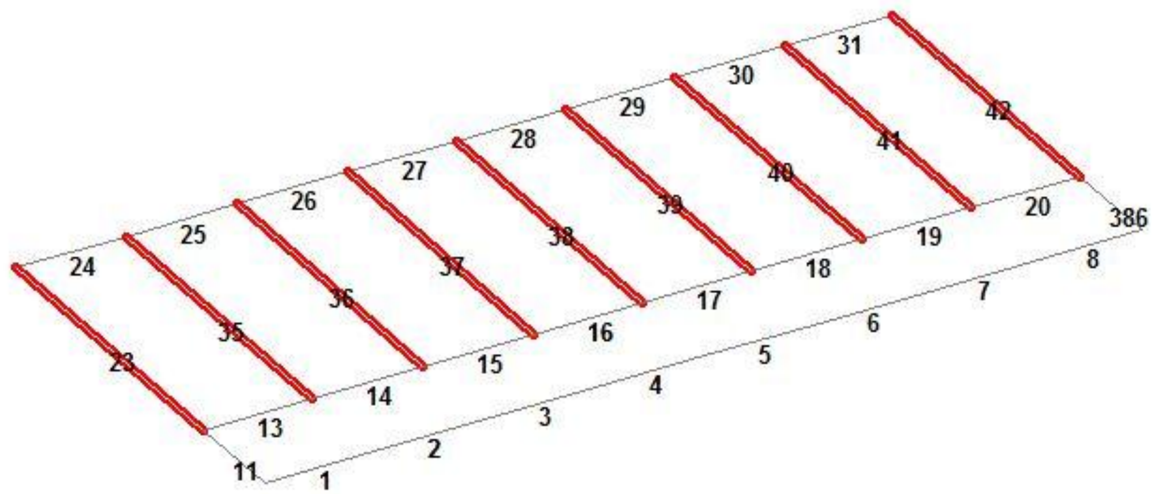


Fig. 3.2 Beams pass in Sagging Moment

Beams Fail in Sagging moment:

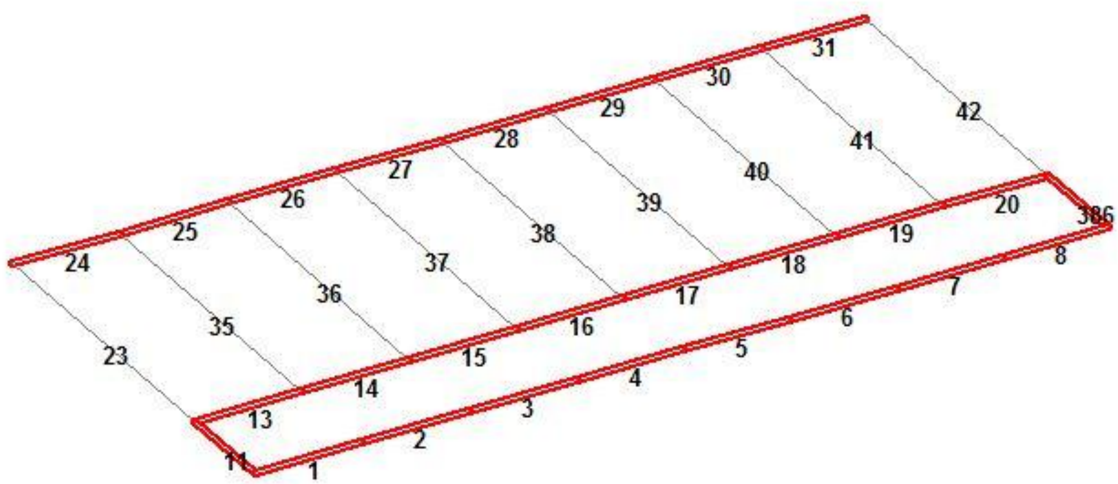


Fig.3.3 Beams Fail in Sagging moment

Beams Pass in Hogging:

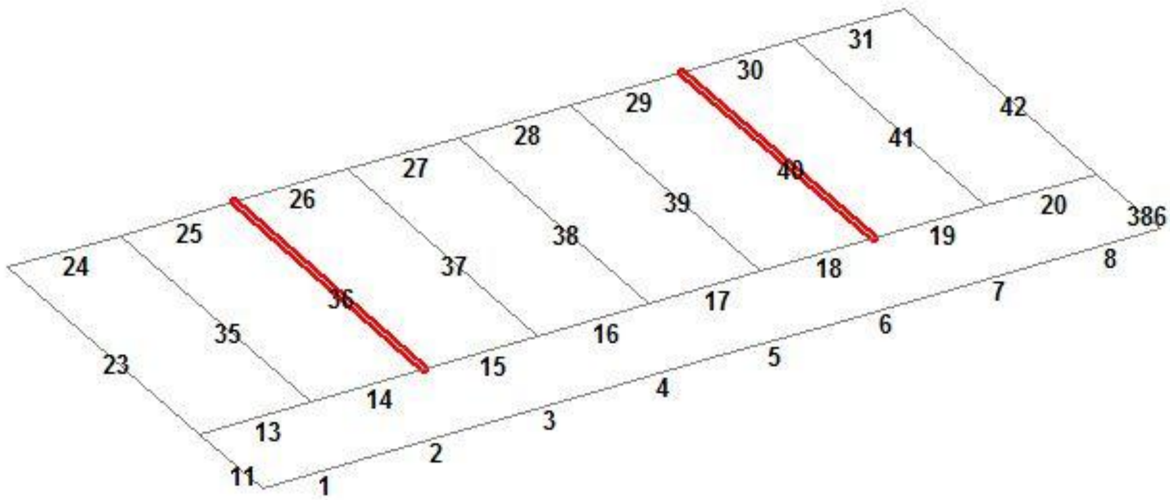


Fig.3.4 Beams pass in Hogging moment

Beams Fail in Hogging:

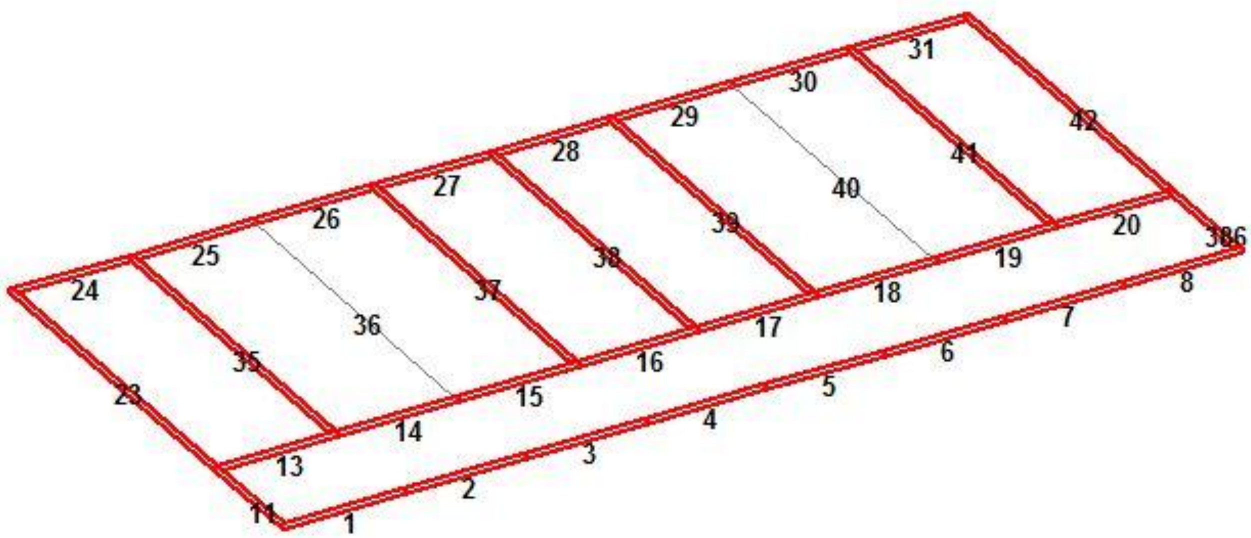


Fig.3.5 Beams fail in Hogging

2. DCR for Shear Capacity in Beams

Shear reinforcement provided in the existing beam at support section is 2 legged 10mm ϕ @140mm c/c.

- We calculate $100A_s/bd$.
- From Table 19 of IS 456:2000 for M15 grade of concrete and value of A_s/bd we find out the respective value of τ_c (Design Shear strength of concrete).
- Using clause 40.4 of IS 456:2000

$$V_{US} = 0.87\ f_Y\ A_{SV}\ d / S_V$$

$$Vu_1 = V_{US} + \tau_c\, bd$$

- We calculate shear from Moment Capacity by using as per IS 13920-

$$V_{U2} = 1.4\ (M_R^{\ H} + M_R^{\ S}) / Lc \qquad L_c = \text{clear span}$$

- Maximum Shear from STAAD.Pro is Calculated.
- Maximum from V_{U1} & V_{U2} is taken i.e. shear resisted.
- If shear resisted is more than maximum shear from STAAD.Pro the member is PASS and Vice Versa.

Sample Calculation For Beam No. 1:

L_C= 3.025m

τ_c = 0.39

Max. Shear from STAAD.Pro = 57.27 KN

V_{U1} = 131.97 KN

V_{U2} = 8.01 KN

DCR= 57.27/131.97 = 0.433 (DCR<1) => PASS.

The results for all other beams are tabulated below:

Table 3.2 Analysis for shear capacity in beams

Beam NO.	Length (m)	Max. Shear (KN)	V_{U1} (KN)	V_{U2} (KN)	Shear Resisted (KN)	DCR	Result
1	3.025	57.278	131.970	8.01	131.970	0.434021	Pass
2	3.051	52.439	131.970	12.44	131.970	0.397354	Pass
3	3.038	52.464	131.970	12.16	131.970	0.397543	Pass
4	2.999	52.069	131.970	12.12	131.970	0.39455	Pass
5	2.975	52.035	131.970	12.26	131.970	0.394293	Pass
6	3.038	52.506	131.970	12.22	131.970	0.397862	Pass
7	3.025	51.974	131.970	12.00	131.970	0.39383	Pass
8	2.965	56.553	131.970	8.204	131.970	0.428527	Pass
13	3.025	102.93	119.133	23.10	119.133	0.863991	Pass
14	3.051	113.103	119.133	26.17	119.133	0.949383	Pass
15	3.038	111.837	119.133	25.30	119.133	0.938756	Pass
16	2.999	106.236	119.133	25.50	119.133	0.891741	Pass
17	2.975	106.308	119.133	25.48	119.133	0.892346	Pass
18	3.038	110.865	119.133	24.99	119.133	0.930597	Pass
19	3.025	112.105	119.133	25.44	119.133	0.941005	Pass
20	2.965	107.247	119.133	24.94	119.133	0.900228	Pass
24	3.025	113.554	152.455	19.00	152.455	0.744836	Pass
25	3.051	113.181	152.455	22.25	152.455	0.742389	Pass
26	3.038	110.244	152.455	20.60	152.455	0.723125	Pass
27	2.999	102.256	152.455	22.32	152.455	0.670729	Pass
28	2.975	102.539	152.455	22.02	152.455	0.672585	Pass
29	3.038	109.94	152.455	20.46	152.455	0.721131	Pass
30	3.025	110.66	152.455	20.99	152.455	0.725853	Pass
31	2.965	104.293	152.455	21.86	152.455	0.68409	Pass
35	8.886	171.364	293.980	21.43	293.980	0.58291	Pass
36	8.886	296.167	285.796	26.98	285.796	1.036287	Fail
37	8.886	170.205	293.980	18.94	293.980	0.578967	Pass
38	8.886	168.774	293.980	18.75	293.980	0.5741	Pass
39	8.886	169.559	293.980	18.78	293.980	0.57677	Pass
40	8.886	295.45	253.308	26.73	253.308	1.166365	Fail
41	8.886	169.327	293.980	19.36	293.980	0.575981	Pass
42	8.886	104.764	198.252	19.40	198.252	0.528438	Pass
23	8.886	231.938	220.076	29.53	220.076	1.053898	Fail
11	2.951	34.446	75.522	6.66	75.522	0.456105	Pass
386	2.951	34.178	75.522	6.61	75.522	0.452556	Pass

Beams PASS in Shear:

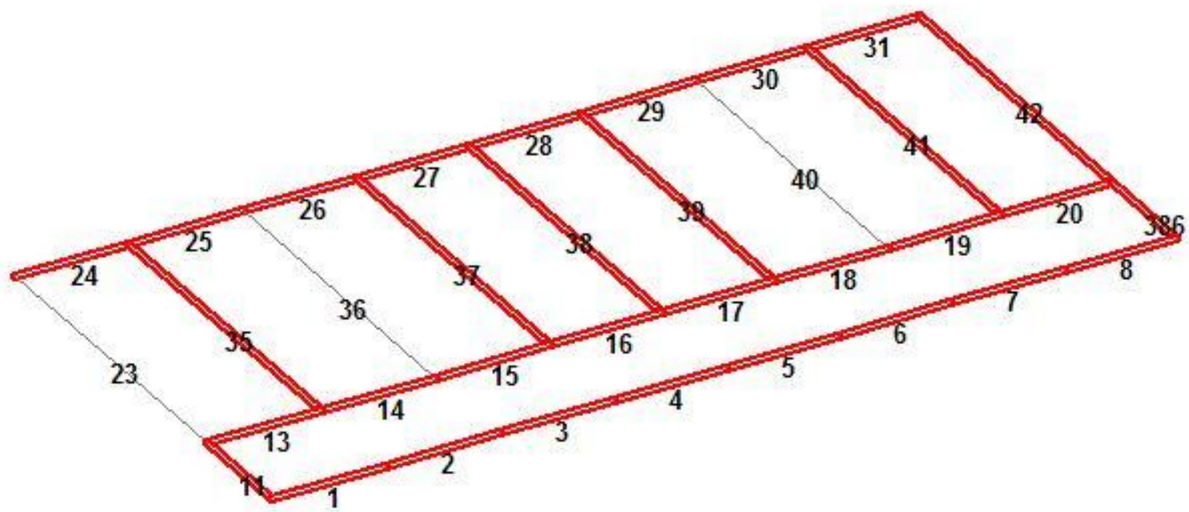


Fig. 3.6 Beams Pass in Shear

Beams FAIL in Shear :

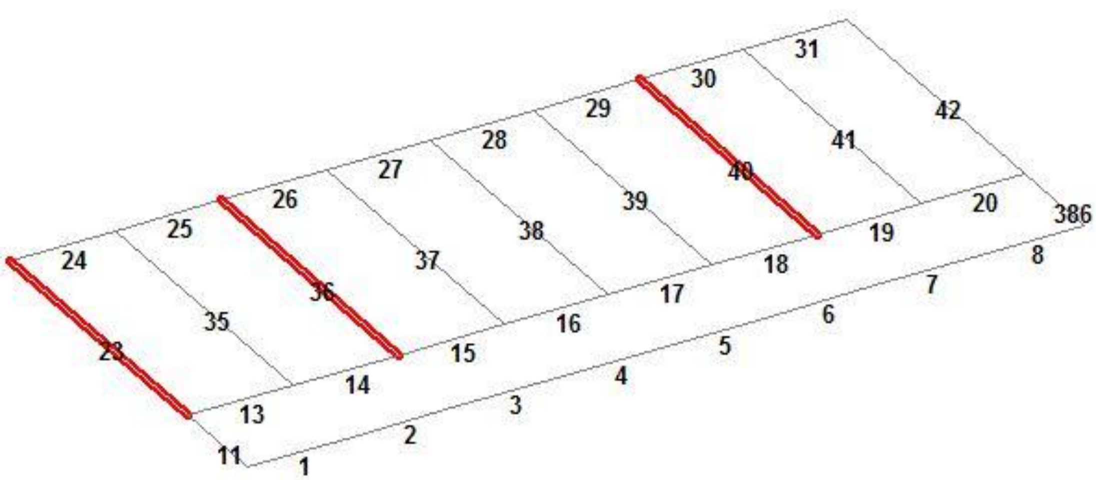


Fig. 3.7 Beams Fail in Shear

3.4.2 Check for Column:

1. DCR for Flexural Capacity of Column :

- The column Demand (M_U) is obtained from Design 2.
- F_{CK} = 15N/mm^2 and f_Y = 250 N/mm^2.
- Cover = 480mm.
- We calculate A_S and then we find out percentage of steel P.
- We calculate P/f_{CK} and P/f_{ck} bD.
- Referring to chart 42 of SP:16, we find the value of M_U'.
- If M_U' is greater than M_U, the member is PASS i.e. DCR < 1.

The results are Tabulated Below:

Table 3.3 Analysis for flexural capacity of columns

Column No.	M_U (KNm)	M_U' (KNm)	DCR	Result
356	42.194	38.27	1.10	FAIL
48	23.63	34	0.69	PASS
124	19.35	29.76	0.65	PASS
200	11.092	25.5	0.43	PASS
367	128.715	153.14	0.84	PASS
59	145.84	102.09	1.42	FAIL
135	128.889	91.88	1.40	FAIL
211	168.413	81.67	2.06	FAIL
375	134.374	171.79	0.78	PASS
67	170.387	160.33	1.06	FAIL
143	152.789	125.97	1.21	FAIL
219	171.695	76.35	2.24	FAIL

Column Fail in Flexure:

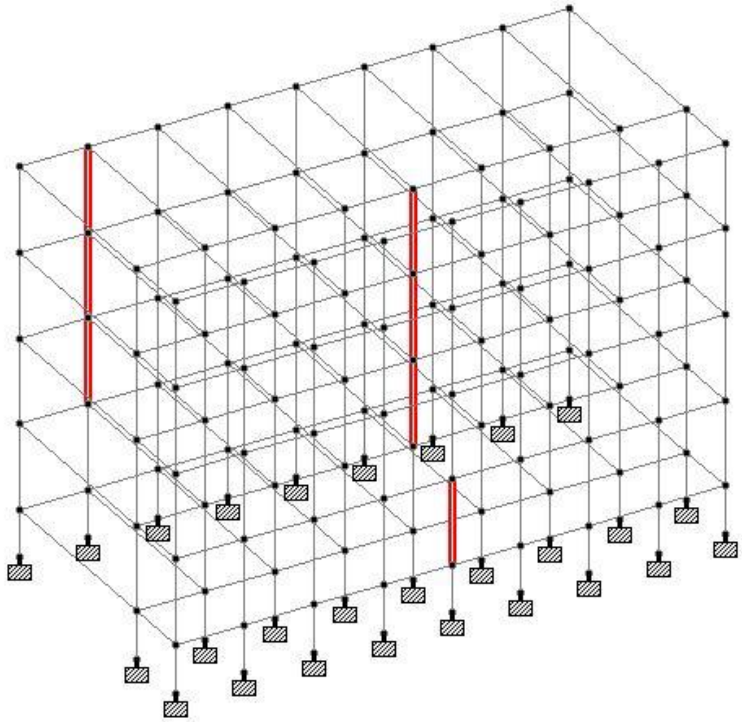


Fig. 3.8 Columns Fail in Flexure

Column PASS in Flexure:

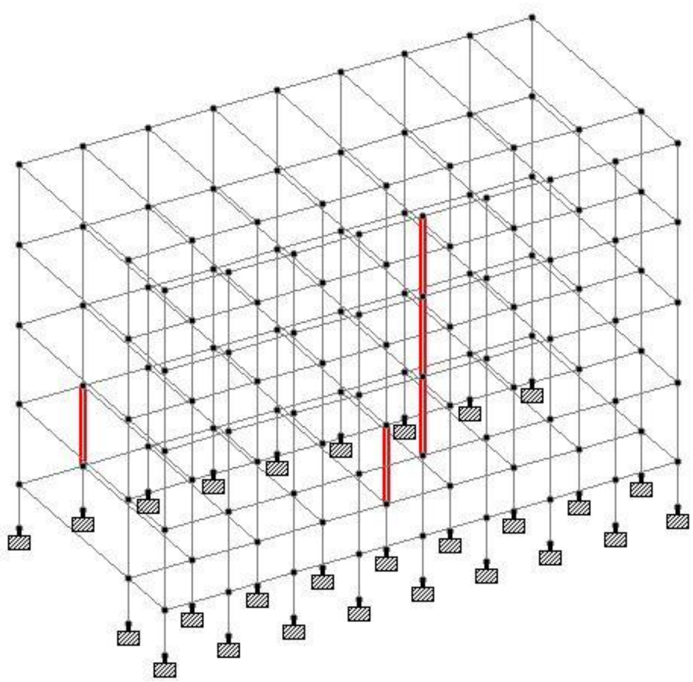


Fig.3.9 Columns Pass in Flexure

2. DCR for shear capacity of column:

- Considering that steel in one face will be in tension and calculated As.
- From Table 19 of IS 456:2000 we get the value of τ_c.
- Stirrups are 4 legged 8mm ϕ @ 190 mm c/c.
- From Clause 40.4 of IS 456:2000 we calculate V_{us} and V_{u1} as done in case of beams.
- Calculated V_{u2} i.e. shear from moment capacity as done for beams.
- Maximum Shear from STAAD.Pro is Calculated.
- Maximum from V_{U1} & V_{U2} is taken i.e. shear resisted.
- If shear resisted is more than maximum shear from STAAD.Pro the member is PASS and Vice Versa.

The results obtained are tabulated below:

Table 3.4 Analysis for Shear Capacity of Columns

Column No.	Max Shear KNm	Shear Resisted (KNm)	DCR	Result
356	31.43	131.6274	0.23878	PASS
48	17.43	131.6274	0.132419	PASS
124	14.27	131.6274	0.108412	PASS
200	10.112	131.6274	0.076823	PASS
367	93.34	208.272	0.448164	PASS
59	119.88	208.272	0.575593	PASS
135	72.367	208.272	0.347464	PASS
211	114.15	208.272	0.548081	PASS
375	100.02	218.3539	0.458064	PASS
67	120.65	218.3539	0.552543	PASS
143	106.10	218.3539	0.485908	PASS
219	118.47	218.3539	0.54256	PASS

3.5 CONCLUSION:

The building is fully analyzed for seismic loads by preliminary and detailed evaluation procedure.

As per the preliminary evaluation of building, the building seems sufficient for earthquakes but this criteria is not enough to conclude any building for its behavior in seismic conditions. Check needs to be done by detailed analysis in order to reach to a concrete conclusion.

The results obtained from detailed analysis shows the deficiency of building towards the earthquake loads. The members may fail in case of seismic activities in future. Demand Capacity Ratio (DCR) is the main key to evaluate a member. If the demand is more than capacity of the member it will obviously fail. DCR values are calculated for Flexural and Shear capacities of beams and columns. For evaluating beams, the first floor beams are taken under study as earthquake load acting on this floor is least among others. Flexural capacity of beams is checked for sagging and hogging moments. The result says almost all beams fail in hogging. Only two beams i.e. 36 and 40 pass in hogging. These are the intermediate beams carrying brick wall. The intermediate beams of classrooms pass in sagging moments and the side beams fails. The beams of corridors fails in sagging and hogging moments. For shear capacity of beams only three beams fail. Again these are the beams which carry brick wall and hence showing behavior different than other beams of same dimension .In case of Columns the ground floor columns of classrooms pass in flexural strength but the ground floor column of corridor fails in flexure. The column at other floors in classrooms fails in flexure and column at other floors of corridor pass. As per the results obtained for shear capacity of columns, all columns pass in shear which shows enough shear reinforcement is present.

As per the results obtained, my evaluation suggests that the frame needs to be strengthened and retrofitted.

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